FEASIBILITY STUDY OF IMPROVED METHODS FOR RIVERBANK STABILIZATION

FINAL REPORT

Sponsored by
PRESIDENT, MISSISSIPPI RIVER COMMISSION

Conducted for
U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
Vicksburg, Mississippi

CONTRACT NO. DA-22-079-CIVENG-63-106

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FOREWARD

This "Feasibility Study of Improved Methods for Riverbank Stabilization" is the final report of investigations made by Harza Engineering Company under Contract No. DA-22-079-CIVENG-63-106, June, 1963, with the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi. The study was sponsored by the President, Mississippi River Commission. The original 12-month contract period was extended to 17 months by supplemental agreements. In addition to brief reports of monthly progress, a more detailed six months progress report was submitted to the Contracting Officer on January 13, 1964.

Dr. James I. Mueller and his staff at the University of Washington (Seattle) assisted in studies of the feasibility of using ceramics. Dr. Daryl B. Simons and his staff at Colorado State University (Fort Collins) assisted in studies of the interdependence of fluvial hydraulics and bank stabilization. Many private and public organizations generously cooperated by supplying information used in this report. Mr. A. R. Tavarozzi of the Union Carbide Plastics Company reviewed most of Part IV, Chapter 2.

GENERAL CONTENTS

SUMMARY

This study was made to investigate new or different materials and methods for protecting the banks of the Lower Mississippi River and to outline future studies needed to clarify the problems involved in such protection.

As provided by the contract, these investigations were made in the following manner:

1. Published reports of research on grouting, soil stabilization, ceramics, chemistry, and other pertinent fields were reviewed to find materials and methods which might be used, now or after further development, to protect riverbanks.

2. Organizations, universities, and individuals having experience with or knowledge of materials and methods which might be used to protect riverbanks were consulted.

3. Representatives of the Mississippi River Commission and the Waterways Experiment Station were consulted for their guidance in determining required performance and characteristics of riverbank protection.

4. Portions of the riverbank were inspected.

The upper concave banks of the Lower Mississippi River are presently protected from erosion by means of quarried stone riprap or uncompacted asphalt pavement. The lower banks are protected by means of articulated concrete mattresses. These are the most effective and economical of the protections yet tried. However, quarry stone is expensive because it must be transported a long distance. The asphalt pavement is expensive to maintain because it does not have sufficient flexibility, permeability, or resistance to mechanical damage. Articulated

concrete mattresses are expensive to build, difficult to place except at low river stages, and replacement costs following bank failures or flanking are high. The articulated concrete mattress may not provide adequate filtering of the foundation.

The lower bank protection represents about 75 percent of the area to be protected and about 90 percent of the total cost of protection. On the basis of cost of materials and placement, the existing lower bank protection is an economical solution when compared to materials and methods investigated. Any effort to improve the effectiveness of the articulated concrete mattress, such as providing filters for the openings, would increase the cost of materials , fabrication or placement of the protection. If the effectiveness of the present protection were better understood, conclusions might be reached which could greatly decrease the overall cost or lead to new concepts of treatment which would be less costly. This understanding can only be reached by an overall study of the present river bank stabilization.

The study would determine the movements of the mattress, the loss of material through the opening between blocks, the length and depth of protection needed, and the effect of the protection on bank failures. This study would of necessity be a broad, long term project closely coordinated with studies of bank stability, channel alignment, river hydraulics, and all related aspects of bank protection. These studies need not include the problems of producing, constructing, and placing the articulated concrete mattresses, but should be directed primarily toward attaining a better understanding of the in-place functioning of the protection.

Many segments of such an overall comprehensive study have been made, such as studies of slide and flow failures. Other studies are now being carried out or planned by the Mississippi River Commission at Vicksburg.

Because of the complexity of river mechanics and river control, a more active interdisciplinary effort to improve knowledge should be encouraged. This effort should integrate the contributions of those working in hydraulics, fluid mechanics, soil mechanics, hydrology, geomorphology, geology, and other related fields.

An extensive program to evaluate the potential of materials and methods to stabilize the lower bank should not be undertaken until an integrated, comprehensive study of all related aspects of bank stabilization indicates that a further effort is justified. The studies needed to clarify the function of the upper bank protection are mainly concerned with drainage and are minor compared to those needed for the lower bank. Although the potential savings in upper bank protection is relatively small since the upper bank involves only about ten percent of the total initial protection cost, the materials which show potential for upper bank protection could be further evaluated at the present time. The use of synthetic materials in sand bags could be further evaluated at present. Sand bags may have many applications in river control works, and research will help river engineers evaluate the properties of synthetic materials for possible future application in lower bank protection.

Upper Bank

On the upper bank, those materials that utilize the natural bank materials show the most potential. Ceramic riprap, formed by the pyroplastic method or by melting sand in an air suspension kiln, shows enough potential to justify keeping abreast of future developments in equipment and techniques.

Soil cement on the upper bank, while it has numerous problems, offers an area of possible economy but would require a carefully carried out program of research and development to actually determine its merits relative to quarried stone or the uncompacted asphalt pavement. Other stabilized soil blocks, such as asphalt and crude glass, may be also worthy of further studies.

Lower Bank

The sand-filled fabric mattress has been used in Holland and Colombia but has never been tried on the Mississippi. In the present stage of development this type protection has many disadvantages, such as deterioration in sunlight, ease of puncture, difficulty in placing and lack of experience in its use on the Mississippi. Also its cost, using materials now available, will be at least slightly higher than the articulated concrete mattress. The fabric mattress may have some advantages over the articulated concrete mattress in that it will adjust to conform more closely to the existing foundation surface and will act as a filter for the underlying soil. The recommended overall river protection studies may well indicate that a more expensive blanket can be economically justified if failure and resulting maintenance costs are reduced by the use of such protection.

Sand bags appear to have potential for use in impermeable deflectors or retards. With the rapid development of cheap plastic fabrics there are now several materials available for the fabrication of sand bags which may find wide application in the future. The most promising are polyethylene-vinyl-acetate copolymer (now manufactured by Union Carbide), nylon, and a specially formulated acrylic (on which development work has been started by Chemstrand, a division of the Monsanto Chemical Company).

PART I

INTRODUCTION, CONCLUSIONS AND RECOMMENDATIONS

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PART I

INTRODUCTION, CONCLUSIONS AND RECOMMENDATIONS

Introduction

The purpose of this study has been to investigate the feasibility of utilizing new materials or methods to stabilize the banks of the portion of the Lower Mississippi River from Cairo to Baton Rouge. The materials and methods were not to be limited to those which could be more economical or more effective than those presently used; but were also to include materials that are in the development stage and those which through technological advancement or mass production might become economical at some future date. It was recognized that in a report of this type in many instances negative conclusions might be as important or more so than positive conclusions. Many materials are described in this report which have little or no use in river control works to prevent the necessity or possibility of future duplication of the investigation into these materials or methods. A detailed study of previous and present bank protection used on the Lower Mississippi River was not made. The protection was reviewed only to the extent necessary to evaluate new methods and materials and to arrive at logical overall conclusions.

Prior to actually starting the studies, a four-day inspection trip of the Lower Mississippi River from Cairo to Vicksburg was made to gain first hand knowledge of the extent and type of river control structure now in use. This trip was made with members of the Mississippi River Commission, the Memphis District, Vicksburg District, Lower Mississippi Valley Division and other Corps of Engineers personnel. Conversations with these experienced river control engineers gave much needed information, not otherwise obtainable, on the experience related specifically to the control of the Lower Mississippi.

A large portion of the studies involved contacting and obtaining information and ideas from many groups who had little or no experience with either the Mississippi River or matters related to slope protection or river control. Therefore, to obtain the maximum understanding in a short time a memorandum entitled "Outline and Design Criteria - Slope Stabilization Methods Feasibility Study" was prepared. The memorandum summarized the present methods of slope stabilization used on the Mississippi River, gave general foundation conditions along the river banks and the soil types in the flood plain, classified the bank treatment and presented the types of treatment to be studied, established design criteria and outlined the studies to be made, and commented on the indirect treatment methods which would be omitted from the study. This outline proved to be invaluable in communicating with those from other professions and interests.

A comprehensive review of bank and channel stabilization methods used on alluvial rivers around the world was made to obtain ideas for new methods and materials.

Several conferences were held with Corps of Engineers personnel in Vicksburg to discuss design criteria, present method and those previously tried on the Mississippi for bank stabilization, and to obtain cost data and pertinent publications from the Waterways Experiment Station library.

The major investigations were as follows:

(1) A study of materials which are either available now or might be available in the foreseeable future which could be used in riverbank stabilization or other related river control structures.

(2) A study of construction and placement methods which have been used, are now used, or could be developed for the purpose of installing bank protection.

(3) A study of the inter-dependence of fluvial hydraulics and bank channel stabilization, and of the criteria which should be satisfied by effective bank protection.

Numerous technical disciplines were investigated during the study. Among these were chemistry (chemical soil stabilizers, polymeric chemistry, asphalt chemistry), physics (electrokinetics, thermodynamics), and others more normally associated with Civil Engineering and Construction such as hydraulics, soil mechanics, grouting, ceramics, and construction materials and procedures.

The primary sources of information used in pursuing this study were (1) available literature and (2) conferences and other communications with industry, educational institutions and trade associations. Using these sources of information, the investigations were primarily directed towards determining the technical feasibility of utilizing different materials and methods. The purpose has not been to find one specific material or method which could be used effectively and economically in river-bank stabilization, but rather to determine whether any of the recent developments in the above disciplines might be applicable thereto or not. Unless the material or method under consideration was obviously so expensive that its use could not be considered reasonable, every attempt was made to first determine the technical feasibility of its use before any economic analyses were made. In most cases such economic analyses are necessarily incomplete or very approximate, because the proposed placement methods have never been used in such an application or on such a large scale. Material costs were estimated as precisely as available information allowed.

No attempt has been made to investigate the mechanisms of riverbank failure (slide or flow) or to investigate methods to prevent or reduce these failures, other than the improved stability resulting from direct and indirect protection. It was beyond the scope of this study to make hydraulic tests in models or prototype testing of the river, nor were special or hydraulic tests made to determine the feasibility of use of the materials or the methods envisioned for riverbank stabilization.

The report is divided into five major divisions as follows:

Part I - Introduction, Conclusions and Recommendations

Part II - Fluvial Hydraulics and Bank Stabilization

Part III - Upper Bank Protection

Part IV - Lower Bank Protection

Part V - Point Bar Protection

Part I, which includes this introduction, introduces the entire report, gives the general conclusions of the study, and makes recommendations for further investigations. Part II includes a discussion of fluvial hydraulics (Chapter 1), bank and channel stabilization methods used on alluvial rivers (Chapter 2), and the design criteria used to evaluate the efficacy of proposed protection methods (Chapter 3). Part III presents a discussion of the physical properties and costs of Upper Bank protection presently in use (Introduction), and summaries of investigations conducted into other types of riprap (Chapter 1) and monolithic (Chapter 2) protection which might be used in protecting the upper banks. Part IV presents a discussion of the physical properties and costs of the articulated concrete mattresses presently used as lower bank protection (Introduction); summaries of the investigations conducted into the possibility of using ceramic mattresses (Chapter 1), synthetic sheeting and fabrics (Chapter 2), and miscellaneous materials (Chapter 3) in revetment type protection of the lower bank; and a discussion of some of the materials which might be used in indirect bank and channel stabilization installations (Chapter 4). Appendix IV-A includes a report on the testing conducted to determine the technical feasibility of using Mississippi River Valley silt and clay deposits or the Vicksburg loess in producing ceramics. Part V presents a discussion of the various methods which might be used in preventing the erosion of the point bars.

A glossary of terms is included in Appendix A. General references that were used in the study are listed in the general bibliography (Appendix B), while references that were used for certain phases of the study are categorized in bibliographies at the end of each Part.

PART I

CONCLUSIONS AND RECOMMENDATIONS

General

In a feasibility study of this type it is not to be expected that the results will permit the drawing of conclusions and recommendations concerning the adoption of a particular method or material for use in river banks stabilization. Rather the result will indicate concepts, areas or directions of future study and availability and possible usage of new materials or methods. To prevent duplication of work in the future and to concentrate the scope of future research and testing, conclusions will also indicate the negative results that were obtained.

Fluvial Hydraulics and Bank and Channel Stabilization on Alluvial Rivers

A better understanding of the functioning of the present protection system is essential to any successful effort to reduce the cost of river bank protection. Also, an understanding of fluvial hydraulics in general and of the condition existing in any reach of river to be stabilized in particular are necessary for a more economical and/or efficacious design of the bank and channel stabilization works.

Unfortunately, the present knowledge of fluvial hydraulics is insufficient to allow a theoretical design of river control works. Rather, design is now more nearly an art with an empirical approach. Therefore, it is concluded and recommended that an extensive effort be made to integrate existing knowledge of the mechanics of flow in open channels, hydrology, geomorphology, soil mechanics and other related fields as they apply to river control structures. A preliminary step in such an interdisciplinary integrated effort would be to compile and rationally interpret the already existing knowledge of fluvial hydraulics and the experiences gained from many years of river stabilization on the Lower Mississippi.

In studying the action of the present revetment, a study of the actual river should also be made. The stream lines, zones of separation and related surface flow phenomena can be studied in the river by using floating lights and cameras to trace the currents. New techniques are being developed using infra-red and regular photographic techniques which indicate that there is a possibility of mapping channel alignment and relief including larger bars and dunes. The development of this technique would enable the river engineer to study in detail the characteristics of a revetment or other river control structure. These techniques, along with the sounding device now being developed for locating the underwater mattress and the possible use of radio active materials in the revetment to trace the movement of the revetment, would help advance the science of river mechanics and river control.

Study of the river should not be limited to the bends but should also include straight reaches with corresponding studies in large scale flumes. The study of the mechanics of flow should give particular attention to the relation between velocity distribution, geometry of cross-section, the bed configuration, formation of large bars and boundary shear stress.

The stream channel forms as a result of a complex interaction of the river, the bed and bank material and the imposed sediment load. The characteristics of both the river and its perimeter change with longitudinal distance and with elevation at a cross-section. The complex feedback mechanism involved makes each river seem independent and unrelated to other rivers. When designing bank stabilization works, it is necessary to understand these phenomena and to be able to take into consideration the variation of both flow phenomena and boundary materials in both vertical and horizontal directions as stage changes.

With the variation of flow phenomena with stage, the forces imposed on the bank stabilization also changes. This is particularly true as conditions change from within channel flow to overbank flow. As overbank flow develops the upper layer of water exerts a large influence on the velocity distribution, boundary shear and sediment transport within the main channel. This new condition brings into play an altogether different set of forces on the bank stabilization works and hence should be understood and incorporated into the design. To improve knowledge of the effect of overbank flow it is essential to initiate a study of this phenomenon.

Statistical methods should be used to analyze and interpret data to establish possible relationships between the hydrograph, both present and past, and the dynamics of the river.

It must be concluded that a further effort to improve modeling techniques should be exerted. Both theoretical model relationships, stemming from continued research into the mechanics of water-sediment flow, and methodology of movable bed models deserve continuous and vigorous study leading to the establishment of model scale relationships for movable bed models.

Also, to a greater extent, models should be used in conjunction with actual river work, not independent of it. That is, the model should be used to check the river and vice versa. By using larger scale models, light weight materials such as crushed walnut shells, plastic particles, crushed bakelite and similar materials it may be possible to study problems without using vertical distortion. This concept is at least worthy of further trial. However, it should be emphasized that the river model if used in conjunction with actual observations and work on the river as a means of qualitatively developing control methods and related structures, should improve our modeling techniques, our understanding of river mechanics and lead to a more economical solution of river problems.

Large rocks, Kellner jetties and other types of roughness have been used effectively to stabilize and control small and medium sized rivers but mostly this procedure has been ineffective on large rivers. It appears that these failures have resulted from using roughness elements too small to hold the main current away from the banks. Research is needed to establish the magnitude of roughness elements required for control of large rivers.

The concept of using the geometry of bends requiring minimum maintenance to revise and stabilize other bends is an important one which is currently used. The river can be subdivided into reaches and within each reach the required modifications in geometry can be patterned after the bends causing the least problems. This procedure compensates for the effect of changes in bed and bank material, flow phenomena and channel geometry with distance, and with further study could be made more effective.

Upper Bank Protection

Both artificial riprap and various forms of monolithic bank protection have been studied for use on the upper bank. Those materials which appear to be technically feasible for such applications are listed in Tables I-1 and I-2.

Artificial Riprap Protection: Of the several types of artificial riprap considered feasible for use in protecting the upper banks, it appears that soil cement blocks, asphalt stabilized soil blocks, crude glass stones and possibly ceramic riprap by the pyroplastic method have the greatest potential for replacing the quarried stone now used.

The soil cement blocks have the lowest cost of raw materials and are technically feasible. However, a carefully planned testing program will be necessary to determine the most economical construction methods. Special attention should be directed toward curing methods, cement contents, durability and methods of breaking up or manufacturing the individual pieces. It may be well to carry out this program in conjunction with a program exploring the use of soil cement in monolithic treatment.

The manufacture of riprap by treating a weak rock with surface coatings, electro-chemical methods, or thermal stabilization does not appear to be technically feasible, nor are there developments which would appear to warrant further study at this time.

The use of commercial ceramic units such as common clay brick and tile or a new product, Poreen, would be technically feasible but the cost is too great using present methods of production and standards for finished products. The use of lower grade raw materials with present production methods would result in some reduction in the cost of these materials, but would also reduce the quality.

The production of ceramic riprap by the pyroplastic method appears to be technically feasible; and the cost of production with low grade raw materials readily available in the river banks, while being approximated to be somewhat higher than the quarried stone, is close enough to warrant further consideration. The development of this process should be followed and an active file on the cost, manufacturing procedures and other pertinent data maintained. Periodic comparisons with the costs of presently used upper bank protection should be made.

The studies indicate that the use of cationic asphalt emulsion in soil stabilized blocks might produce a riprap type protection which would be somewhat less expensive than soil cement but slightly more expensive than those methods now in use. Further study of the relative thickness of protection required for the asphalt stabilized block compared to the riprap protection, durability and length of life and the filter requirements with the asphalt block could yield a more favorable economic advantage to the asphalt block. The asphalt-sulphur hot mix stabilized blocks are more expensive, but they have some technical advantage such as shorter cooling time and ease of handling. The cost of raw material is the largest drawback to this treatment.

The production of crude glass riprap (melting sand) appears to be technically feasible. The present state of knowledge of the methods and equipment which could be used in its production and the resulting quality of the finished product are not sufficient to allow a reasonable estimate of the cost of producing riprap. An effort should be made to keep abreast of future developments of equipment and methods of production.

Chemically stabilized soil blocks are technically feasible, but chemicals are too expensive at this time. Any future study of these blocks for riprap should be restricted to keeping abreast of developments in chemical soil stabilizers, although the future does not look promising.

Monolithic Protection: None of the types of monolithic upper bank protection considered appear to have exceptional potential at present when compared with the uncompacted asphalt pavement. Those which show the most potential for further study are soil cement, synthetic elastomer sheeting, and the use of new asphalt materials in the uncompacted asphalt pavement.

When the drainage problem on the upper bank, stability of the banks, the function of the upper bank protection during overbank flow, and other requirements for the protection are more clearly defined, it is possible that some of the methods investigated will have more potential than they appear to have now. At present, their potential is restricted more by the drainage problem than other requirements.

Soil cement has proven to be effective in protecting earth dams, but needs to be investigated further to determine if a thinner protection will be effective and economical on the river bank. Synthetic elastomer sheeting appears to be economical when considering in place cost, but it needs to be further investigated to determine its economy considering maintenance and to determine its effectiveness at a time when the function of upper bank protection is more clearly defined. Other synthetic materials should not be used on the upper bank until their resistance to actinic deterioration is considerably improved.

Cationic emulsion asphalt, asphalt-sulphur mixes, and Plasmofalt have specific properties that might be used to further improve the uncompacted asphalt pavement. Their potential for such use needs to be further evaluated by making a detailed comparative economic study with the present hot-mix uncompacted asphalt pavement. Application of asphalt or asphalt-sand slurry by spraying is not economical at present. Prefabricated asphalt mattresses do not offer any technical or economic advantage over the uncompated asphalt pavement.

Chemical soil stabilization and metal sheeting are too expensive, and the quality of chemically stabilized soil protection is doubtful. Unless the basic material cost is considerably reduced, which is not likely to happen, these methods do not warrant further investigation. The economic and perhaps technical potential of thermal stabilization must be evaluated after equipment is developed that can apply heat efficiently for such a purpose. Not enough is known about the quality of such protection to justify developing equipment for this specific purpose. Therefore the potential of this method cannot be properly evaluated until more is found out about the quality of such protection from laboratory testing or until industry develops equipment that appears to warrant field testing.

Lower Bank Protection

The investigations which have been conducted on the possibility of using new materials or methods for protecting the lower banks have been primarily concerned with revetment type protection. Those methods and types of materials which have been found to be technically feasible for use as direct protection are listed in Table I-3. Consideration has also been given to materials which could be used in indirect protection methods, although the effectiveness of such installation depends more upon hydraulics considerations than on the materials used in their construction.

Direct Protection: Of the numerous materials considered for use in revetment type protection, it appears that the synthetic fabric probably have the greatest potential for providing efficacious and economical bank protection. There are many different such materials available, however only one, nylon has been used in riverbank protection installations. In the forms now available nylon is not sufficiently resistant to deterioration from sunlight to allow its use above the low water level. Acrylics are more resistant but are not as strong as the nylon fabric.

It is believed that these materials when used in the form of sand filled mattresses will provide bank protection superior to that now used in terms of:

1. Filtering properties
2. Flexibility
3. Anchoring

The sand filled mattress will be equivalent to the articulated concrete mattress in terms of permeability, but it may be inferior in terms of placement conditions and resistance to mechanical damage. It is possible to use this method in conjunction with an articulated concrete mattress with a filter in the upper portion of the bank where mechanical damage is most likely and where the fabric would be subjected to sunlight.

Of the placement method considered, Barge Placement Method-II appears to have the most potential for the following reasons:

1. Complete filling of the mattresses is ensured.
2. The operations of fabricating, filling and placing the mattresses is continuous and practically automatic.
3. The size of the mattress required can be determined at the time of launching.

It should be pointed out that the placement methods given herein are only ideas and are presented as a starting point for consideration after the material included has been proven out by test and trial sections.

Concerning the other materials and methods listed in Table I-3, the following conclusions have been drawn:

The commercial ceramics, Poreen and hot-press ceramics (pyroplastic method) would all be expensive to fabricate into mattresses. Both the Poreen and the commercial ceramics, even without fabrication cost, would be too expensive. The Poreen has strength and durability qualities greater than actually needed for revetment purposes. The cost of the hot-press ceramics lies mostly in the cost of production equipment. The development of this process and equipment should be followed as recommended previously under upper bank riprap protection.

With the present knowledge of the function of the revetment type protection it is not possible to come to a definite conclusion on the use of a single layer synthetic sheeting or fabric for lower bank protection. This type protection would have many placement and anchorage problems. It is not, however, removed from the realm of possibility and should be kept in mind as research progresses.

The prefabricated asphalt mattress presents placement problems due to light weight, difficulty of adding sufficient holes to allow ease of sinking the mattress and the susceptibility of the asphalt material to creep giving both anchorage problem and deterioration in place as erosion proceeds at the thalweg. The prefabricated asphalt mattress also presents a problem in filtering the holes and preventing excess uplift.

Metal sheeting or metal fabric mattresses are too expensive to allow serious consideration. These mattresses would also entail placement and handling problems with the same filtering problems as the prefabricated asphalt mattress.

Both riprap and the placement of mass asphalt underwater were considered. It would be impossible to insure continuous permanent coverage with either of these methods without using excessive amounts of material at an unjustifiable cost.

With the material available and the placement problem involved it does not appear that a gabion type protection can be either technically feasible or economical.

Indirect Protection: The concept of the use of submerged retards should be studied both in the laboratory and in the river as outlined earlier in these conclusions and recommendations. Until the roughness scale criteria has been established it would be open conjecture to try to predict the most suitable type of material for these submerged retards.

There are several synthetic materials which appear to have the necessary properties for use in fabricating sand bags for use in impermeable deflectors. The materials which are most promising are nylon, for uses when the material is always submerged; acrylics, where somewhat less strength can be tolerated; and one of several plastic sheetings, the most promising of which is polyethylene-vinyl-acetate copolymer.

A program of testing sand bags should be formulated and copies sent out to the various interested industries to give a more comparable basis for considering the various materials. The tests could include, tensile strength, tear strength, bursting strength of a particular size of bag, effects of various percentages of filling, the sliding resistance of the bags, bag to bag contact and bag to foundation material contact.

Point Bar Stabilization

With the present knowledge of the need and required location for point bar stabilization it is not possible to recommend a treatment for this purpose. It does, however, appear that with the problem of anchorage, erosion resistance, need for very low cost, and necessity of application both above and below the water level that the only material that is worthy of further consideration at this time is cationic asphalt emulsion.

Table I-1

RIPRAP TYPE UPPER BANK PROTECTION

No.	Material	Estimated Cost of Material per ton of Riprap 1/	Application	Estimated Cost of Processed Materials per square foot 2/	Estimated In Place Cost per square foot 3/	Remarks
1.	Quarried Stone		10 in. thick layer dumped stone		$0.19 6/	Presently being used.
	Ceramics					
2.	Commercial Brick	$40.00	10 by 10 foot mats, 4" thick	$0.55	$0.65	Individual bricks are too light for dumped riprap.
3.	Commercial Tile	$20.00	10 by 10 foot mats, 4" thick	$0.40	$0.50	Individual tile are too light for dumped riprap.
4.	Poreen	$15.00	10 by 10 foot mats, 3" thick 12" by 24" blocks - hand placed	$0.35 $0.31	$0.48 $0.71	Too expensive for dumped riprap. 60 to 70 lb. pieces with interlocking features.
5.	Hot Pressed Blocks	$ 2.60 4/ $ 4.70 4/	10 by 10 foot mats 4 ton per square - dumped	$0.07 $0.19	$0.20 $0.26	Technical feasibility uncertain. Based on a 50% size of 70 pounds.
6.	Melted Sand	$ 1.60 5/	Crude glass heated in air-suspension kiln			Technical feasibility uncertain.
	Stabilized Soil Blocks					
7.	Soil Cement	$ 1.75	4 ton per square - dumped	$0.30	$0.37	Based on a 50% size of 70 lbs.
8.	Hot-mix Asphalt	$2.00-$2.75	4 ton per square - dumped 5 in. layer of hand placed blocks	$0.51 7/ $0.30	$0.58 $0.70	Based on a 50% size of 125 lbs. 50 pounds per square foot.
9.	Cationic Emulsion Asphalt	$4.50-$6.25	4 ton per square - dumped 5 in. layer of hand placed blocks	$0.43 7/ $0.25	$0.50 $0.65	Based on a 50% size of 125 lbs. 50 pounds per square foot.
10.	Asphalt-Sulphur	$3.50-$4.75	4 ton per square - dumped 5 in. layer of hand placed blocks	$0.41 7/ $0.24	$0.48 $0.64	Based on a 50% size of 125 lbs. 50 pounds per square foot.

Notes: 1/ Transportation costs of commercial ceramics and stabilizers for soil blocks are not included in the estimates.
2/ Includes all material costs and cost of producing hot pressed blocks and stabilized soil blocks.
3/ Includes all anticipated costs except grading of the bank, the cost of a filter blanket, and the contractor's profit.
4/ Cost of producing blocks, including no overhead or profit.
5/ Estimated fuel cost at 50 percent fuel efficiency.
6/ Includes transportation to Vicksburg.
7/ Cost not proportional to hand placed blocks due to forming and production rate.

Table I-2

MONOLITHIC UPPER BANK PROTECTION

No.	Material	Application Method	Durability and Strength	Flexibility	Permeability and Filtering Properties	Placement Conditions, etc.	Estimated Raw Materials Cost per sq. ft.	Estimated In Place Cost per sq. ft.	Remarks
1.	Asphalt-Hot Mix	Premixed and spread	F	F	F	F	$0.03	$0.22	Presently being used.
2.	Soil Cement (Regular)	Premixed, spread, and compacted	G	V.P.	P 3/	F	$0.04	$0.15-$0.20	
3.	Soil Cement (Regular)	Mixed and compacted in Place	G	V.P.	P 3/	P	$0.04	$0.15-$0.20	
4.	Plastic Soil Cement	Premixed and spread	F	V.P.	P 3/	F	$0.06	$0.12	3-inch thickness.
5.	Cationic Asphalt Emulsion	Premixed and spread	F	F-G	F	F-G	$0.06	$0.20-$0.25	
6.	Cationic Asphalt Emulsion	Premixed and sprayed	F	F-G	F	G	$0.06	$0.42	Technical feasibility uncertain.
7.	Cationic Asphalt Emulsion	Sprayed	F	F-G	P-F	G	$0.06	$0.30-$0.40	Technical feasibility uncertain.
8.	Thermal (Fuel Oil)	Melted in Place	V.P.-G	V.P.	P-F	P-G	$0.06 4/	-	Technical feasibility uncertain.
9.	Thermal	Premelted in Kiln	V.P.-G	V.P.	P-F	P-F	$0.03 4/	-	Technical feasibility uncertain.
10.	Synthetic sheeting or Fabric		V.P.-G	G	G	F-G	$0.03-$0.10 5/	$0.15-$0.25	Anchored with soil anchors.

Keys: V.P. = Very Poor
P = Poor
F = Fair
G = Good
V.G. = Very Good

Notes:
1/ Only includes cost of materials which have to be transported to the placement site, based on a 5 inch thickness.

2/ Includes processing and placement costs.

3/ Adequate permeability could only be provided by forming weep holes after placement, or by under drainage blankets.

4/ Fuel Oil Cost.

5/ Synthetic Material only.

Table I-3

LOWER BANK REVETMENT PROTECTION

No.	Material	Placement Method	Durability	Strength	Flexibility	Permeability and Filtering Properties	Weight or Anchorage	Placement Conditions	Estimated Raw Material Cost per sq. ft.	Estimated Placement Cost per sq. ft.	Estimated In Place Cost per sq. ft.	Remarks
1.	Articulated Concrete Mattress	Barge	G	250 lb/in.	G	G-P	20 lb/sq. ft.	F-G	$0.186 1/		$0.40	Presently being used.
2.	Commercial Ceramic Mattress	Barge	G	H	F-G	G-P	20 lb/sq. ft.	F-G	$0.20 2/			
3.	Poreen Mattresses	Barge	G	H	F-G	G-P	20 lb/sq. ft.	F-G	$0.25 2/			
4.	Hot-Press Ceramic Mattress	Barge	G	H	F-G	G-P	25 lb/sq. ft.	F-G	$0.032 3/			
5.	Synthetic Sheeting Mattress	Barge or Roll	P-G	M	E	G	20 lb/sq. ft.	F-G		$0.08-$0.13		
6.	Synthetic Fabric Mattress	Barge or Roll	P-G	M	E	G	20 lb/sq. ft.	F-G	$0.25-$0.40 4/	$0.08-$0.13	$0.65-$0.75 7/ 9/	
7.	Synthetic Sheeting or Fabric	Roll	P-G	M	E	G	P-F	F-G	$0.05-$0.15 5/	$0.05	$0.18-$0.30 9/	Secured with soil anchors.
8.	Prefabricated Asphalt Mattress	Barge or Roll	G	L-M	G	P-G	F	F-G	$0.42 6/	$0.16 8/	$0.58	

Key:
- V.P. = Very Poor
- P = Poor
- F = Fair
- G = Good
- V.G. = Very Good
- E = Excellent
- L = Low
- M = Medium
- H = High

Notes:
1/ See Table IV-1.
2/ Transportation costs of ceramic materials are not included.
3/ Includes total cost of producing blocks except overhead.
4/ Single thickness of synthetic material times 2.74.
5/ Single thickness of synthetic material only.
6/ Total cost of fabricating 5 inch thick mattress with reinforcement.
7/ Based on Nylon at $0.10 per square foot of material.
8/ Placement cost includes fabrication.
9/ In place cost includes raw material, fabrication, and placement.

PART II

FLUVIAL HYDRAULICS AND BANK STABILIZATION

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PART II

FLUVIAL HYDRAULICS AND BANK STABILIZATION

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PART II

FLUVIAL HYDRAULICS AND BANK STABILIZATION

Introduction

The problem of river control for flood protection and navigation dates back to antiquity. The methods which have evolved are the result of many years of experience of river engineers. These methods are based largely upon experience and intuition, an art, whereas a more scientific approach utilizing the fundamentals of science is urgently required if development of better methods is to be realized. That is, the concepts and procedures should become more deterministic and less artistic.

One of the fallacies of not treating river problems as fundamentally as possible is that often misconceptions are perpetuated which are expensive and misleading. Certainly a more scientific procedure would simplify the training of specialists and minimize the danger of losing ground with the changing of staff. Furthermore, such an approach is the only realistic forthright procedure which is capable of giving continuous improvement of techniques, methods and materials applicable to river control.

Upon recognizing the need for a more deterministic approach to river control methods, it became apparent that bank protection methods also needed such an approach; for bank protection is primarily a method of river control. Although a comprehensive review of all the disciplines that might be applied to riverbank protection was beyond the scope of this study, a study of the application of fluvial hydraulics to bank and channel stabilization and a study of stabilization techniques used on other alluvial rivers was made.

The present status of the mechanics of flow in alluvial channels was reviewed to establish what is now known about the subject, to determine what is presently known that can now be used in riverbank protection, and to provide a basis for determining what additional studies are needed to develop a better understanding of the function of riverbank protection.

Stabilization techniques used on alluvial rivers throughout the world were studied and evaluated to determine if any were applicable for use on the Lower Mississippi. Some of the methods of applying the theory of mechanics of river flow to the design of indirect protection were considered.

In order to evaluate the potential of materials and methods for riverbank protection, it was necessary to establish criteria that could be used to judge the effectiveness of the proposed protection. The set of criteria that was used for this study was developed from present knowledge of methods of riverbank protection on the Lower Mississippi River. It defines the properties that the protection material should have, the required behavior of the protection, and the placement conditions. However, these criteria do not assure the effectiveness of protection which meets all of the requirements. Other factors influence or determine the effectiveness of the protection, such as slope stability, the interdependence of fluvial hydraulics and bank protection, or the function of the protection in the overall river control scheme. Thus, the design criteria used for this study should be altered and expanded by future studies that consider all of the related aspects of riverbank stabilization.

PART II

Chapter 1

THE MECHANICS OF FLOW IN ALLUVIAL CHANNELS

General

The mechanics of rivers are very complex. The many variables which are involved are interrelated and in many instances are difficult to measure. Nevertheless there is an excellent opportunity to contribute significantly to knowledge of this problem if detailed theoretical, laboratory and field studies are initiated and coordinated.

Until the knowledge of river mechanics is further developed it would be presumptuous to give detailed advice to trained river engineers. On the other hand, much effort has been devoted to the study of open channels in recent years. Undoubtedly immediate and positive results could result from an interdisciplinary attack on the problem using the field experience of river engineers and the knowledge of the scientists including experts in the fields of fluid mechanics, hydraulics, hydrology, soil mechanics, soil physics and other related fields. An excellent presentation of the present status of river mechanics was written by Rzhanitsyn (Ref. II-14). Much of the theory pertaining to rivers presented in this chapter was taken therefrom.

Characteristics of Rivers

A river net consists of a live, dynamic system of streams which results from a complex interaction between:

1. Physical and geographical conditions which determine the magnitude and distribution of water to the drainage basin.

2. The hydrological factors which affects the runoff of the water and sediment.

3. The geomorphological factors such as the resistance of the earth's surface and streams to erosion.

The stream channels vary in geometry and capacity throughout their lengths. In general the stream gradient flattens, the particle sizes of the bed and bank material decrease, the width-depth ratio increases, and the discharge increases with distance. The variation of these characteristics is in no way uniform. Stream characteristics change radically along a channel, such as at the confluence of the major stream and its tributaries or where the characteristics of the bank material change. For example, the Mississippi River narrows at its lower end. This results from a reduction in discharge and the occurrence of a finer, more stable bank material.

In order to develop a better appreciation of the present knowledge of river mechanics which should be considered when working with rivers it is necessary to: (1) define the forms of bed roughness resulting from the complex interaction between the flow and channel boundary material, and (2) discuss the regimes of flow and forms of bed roughness as observed and studied by Simons and Richardson (Ref. II-18) and others.

These forms of bed roughness are for idealized flow in straight reaches of uniform alluvial channel. In straight reaches of natural rivers the cross-section and flow are non-uniform resulting in non-uniform boundary shear and different bed roughness at areas subjected to different shear stress. That is, the bed may be plane in the vicinity of the thalweg and have dunes adjacent to the plane bed (multiple roughness). A still further limitation is that, for alluvial rivers, straight reaches are the exception. Rivers formed in alluvial material usually consist of a series of bends whose radii and lengths are a function of many variables.

Regimes of Flow in Alluvial Channels

The flow in alluvial channels can be classified into lower flow regime and upper flow regime, with a transition between. This classification is based on similarities of form of the bed configuration, mode of sediment transport, process of energy dissipation, and phase relation between the bed and water surface. The typical forms of bed roughness are illustrated in Figure II-1.

This subdivision into flow regimes is quite general. There probably are an infinite number of different roughness heights and patterns within any single subdivision such as ripples, ripples on dunes, or dunes. But, since within any one of these suggested categories the shape and spacing of the elements, the resistance to flow, and sediment transport are similar and also quite different than in other categories, the proposed subdivision of bed forms is a convenient way to describe form roughness and hence resistance to flow in a general way.

Lower Flow Regime: Most sand bed streams flow in lower regime except possibly at flood stage and in the deeps adjacent to the concave banks. In the lower flow regime resistance to flow is large and sediment transport small. The bed form is either ripples or dunes or some combination of ripples and dunes which are triangular shaped irregular elements. The water surface undulations are out of phase with the bed surface, and there exists a relatively large separation zone downstream from

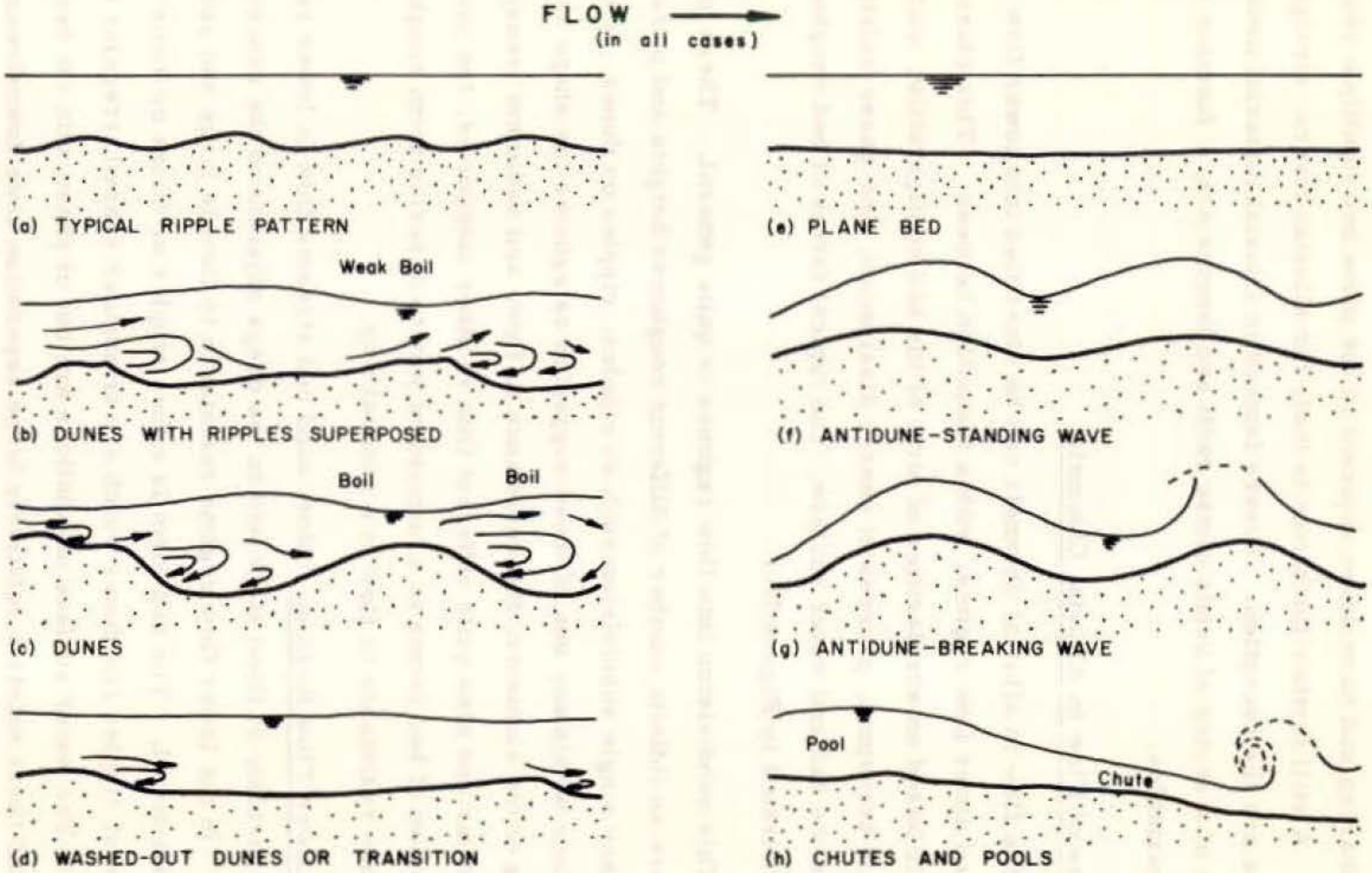


Figure II-1. FORMS OF BED ROUGHNESS IN ALLUVIAL CHANNELS the crest of each ripple and dune. The amplitude and spacing of the large dunes are very responsive to minor changes in the many variables which affect them. Total resistance to flow is the result of grain roughness and form roughness with the latter dominant. The most common mode of bed material transport is for the individual grains to move up the back of the ripple or dune and avalanche down its face. After coming to rest on the downstream face of the ripple or dune, the particles remain there until exposed by the downstream movement of the dunes, whereupon the cycles of moving up the back of the dune, avalanching and storage are repeated. In flume studies the ripple bed configuration is the usual one studied within the lower flow regime; whereas, in natural streams and rivers, dunes or dunes with ripples superimposed are the dominant bed forms in the lower flow regime.

Upper Flow Regime: Usually upper flow regime is observed when a river with a sand bed, particularly fine sand, is at or near flood stage. In the upper flow regime resistance to flow is small and sediment transport is large. The usual bed forms are plane bed or antidunes. The water surface is in phase with the bed surface except when an antidune breaks, and normally there is no separation between the fluid and the boundary. The mode of sediment transport is for the individual grains to roll almost continuously downstream in sheets a few diameters thick, although when antidunes break much bed material is briefly suspended; then movement of water and sediment very nearly stops temporarily; the suspended material falls toward and onto the bed; and there is some storage of the particles that reach the bed prior to re-establishment of flow.

Transition: The bed configuration in the transition from the dunes of the lower flow regime to the plane bed and standing waves of the upper flow regime is erratic. In the transition between the two flow regimes, the bed configuration may range from that typical of the lower flow regimes to that typical of the upper flow regimes, depending primarily on antecedent conditions. If the bed configuration is dunes, the depth or slope can be increased to values more consistent with the upper flow regime without changing the bed form; or conversely if the bed is plane, depth and slope can be decreased to values more consistent with dunes without a change in the bed form. Often in the transition from the lower to the upper flow regimes, the dunes will decrease in amplitude and increase in length before the bed becomes plane (sheared-out dunes). Needless to say, change in depths, resistance to flow, and sediment transport have all the variability of the bed configuration in the transition.

Forms of Bed Roughness and Flow Phenomena

Under idealized conditions such as in a flume or uniform straight reach of a sand bed river, the order of occurrence of the forms of bed roughness with increasing stream power, $V\tau_o$, (where V is the mean velocity, $\tau_o = \gamma DS$ is the shear stress, is the unit weight of water, D is depth of flow, and S is the slope of total head-line)starting from plane bed without movement, is ripples, ripples on dunes, dunes, transition, plane bed, antidunes, and chutes and pools for bed material finer than 0.6mm. For material coarser than 0.6mm dunes form after beginning of motion at small values of stream power instead of ripples. In the following sections, these bed configurations and the associated flow phenomena are described in the order of their occurrence with increasing stream power.

Ripples: Ripples are small elements triangular-shaped in profile with gentle upstream slopes and steep downstream slopes. They are less than about 1.5 feet in length, and 0.02 foot to 0.2 foot in height, and have rather small width normal to the direction of flow. When initially formed, ripples are quite uniform in shape and spacing, but with continued flow the spacing becomes more irregular and the ripples may be considered of uniform size and shape only in a statistical sense. Resistance to flow is large, $C/\sqrt{g}$ ranging from 7 to 12, where C is the "Chézy Coefficient." There is a relative roughness effect with resistance, decreasing with increasing depth. Suspended bed material load is small. If there is no wash load the water is clear enough that the bed configuration can be photographed through the running water. The concentration of total bed material discharge is relatively small, ranging from 10 to 200 parts per million (ppm), by weight.

Dunes: If by one means or another, usually an increase in depth or slope, one gradually increases the boundary shear stress or stream power associated with ripples or the plane bed in the case of coarse sand bed materials, then a rate of bed material transport, a magnitude of velocity and a degree of turbulence are soon achieved that causes larger sand waves, called dunes, to form. With the smaller shear stress values, the dunes will have ripples superimposed on their backs. These ripples will disapper at larger shear values particularly with the coarser sand, $d_{50} > 0.4$ mm.

The dunes are large elements triangular-shaped in profile similar to ripples. Their lengths range from two feet to many feet and heights from 0.2 feet to many feet depending on the scale of the flow system. In the large flume (described in Ref. II-17), the dunes range from two feet to ten feet in length and from 0.2 to 1 foot in height; whereas Carey and Keller (Ref. II-3) gave lengths of dunes for the Mississippi River of several hundred feet and heights as large as 40 feet. The maximum amplitude to which the dunes can develop is approximately average depth of flow above the portion of the bed in question. Hence, in contrast with ripples, the amplitude of the dunes can increase with increasing depth so that the relative roughness can remain essentially constant or even increase with increasing depth of flow. Even normal changes in water temperature or concentration of fine sediment can significantly affect the magnitude of the dunes and hence depth and velocity distribution in sand bed channels.

Field observations indicate that dunes will form in any channel irrespective of the size of bed material if the stream power is sufficiently large to cause general transport of the bed material without exceeding a Froude number of unity. Furthermore, the length and shape of the dunes are a function of the fall velocity of the bed material although dune height did not appear to be in the flume experiments conducted at Colorado State University. The length of the dunes increased and the angle of the upstream face and downstream face decreased with a decrease in fall velocity. That is, with fine sand the length of the dunes is greater, and their shape is less angular than for the coarser sands.

Resistance to flow with dunes is large; although in the flumes (described in Ref. II-17) with the finer sands and limited depth, resistance to flow for ripples was larger than for dunes. As a result the discharge coefficient, $C/\sqrt{g}$, varied from 8 to 12. With changes in depth and fall velocity, reistance to flow increased with depth for coarser sands and decreased with depth with the finer sands. The dunes cause quite a disturbance in the flow. The separation zone is very large, causing large boils on the surface of the stream. Therefore, the flow contains relatively large concentrations of suspended sediment and the concentration of the total bed material discharge ranged from 200 to 2000 ppm. Measurements of velocities within the zone of separation showed that upstream velocities existed that were one-half to two-thirds average stream velocity and the boundary shear sometimes was sufficient to form ripples oriented opposite to the direction of the primary flow in the channel.

Some investigators do not agree that there is a difference between ripple and dune bed configurations. This question might appear academic were it not for the fact that in most flume investigations the ripple bed dominates, whereas in the field, dunes dominate. As Taylor and Brooks pointed out, if there is a fundamental difference between ripples and dunes, the problem of roughness analysis and modeling of alluvial channels cannot be easily resolved by small scale laboratory studies (Ref. II-20).

Plane Bed with Sediment Movement: A plane bed is a bed without elevations or depressions larger than the maximum size of the bed material. The resistance to flow with the plane bed results largely from grain roughness and $C/\sqrt{g}$ is large, ranging from 14 to 23. However, the grains are rolling, hopping and sliding along the bed, and the resistance coefficient is slightly larger than for the static plane bed. A large part of the transported bed material is confined to a relatively thin layer near the bed and the total bed material transport concentration ranged approximately from 2000 to 6000 ppm. With a change from dunes to plane bed a large reduction in stage may occur as has been observed on many rivers.

The magnitude of the stream power ($\tau_0 V$) at which the dunes or transition roughness changed to the plane bed depended primarily on the fall velocity of the bed material. With the fine sands (low fall velocity) the dunes washed out at relatively low stream power and small Froude number in comparison to the coarser sands. In natural streams with large depths the change from transition to plane bed occurs at much lower Froude numbers than in the flume experiments because of the larger stream power (large depth).

Antidunes: Antidunes are in-phase (coupled), symmetrical sand and water waves. The height and length of these waves depends on the scale of the flow system and the characteristics of the fluid and bed material. In the Rio Grande, surface waves from two to three feet in height and 10 to 20 feet long have been observed (Ref. II-12). Antidunes are not common to our large rivers except under very special and localized conditions.

Large Bars: The development and movement of the large alternate bars that form at large width-depth ratios, and that have been observed both in the flume at Colorado State University and in the field, present especially interesting complications in the structure of bed features. As these bars form and migrate downstream, the velocity, depth, and direction of flow change with time and with distance along the bar. Smaller bed features, the ripples, dunes, and plane bed, may all be present simultaneously on the bar; and on the downstream part of the bar where deposition is occurring, the depositional sequence gives a reasonable indication of the flow regime associated with the deposition. However, as the direction of flow over the bars changes, so does the orientation of the smaller bed forms; and in any given depositional sequence, a variety of orientations present themselves, perhaps none of which indicate a direction of flow parallel to the main current of the stream.

The development and subsequent erosion of large alternate bars relates significantly to: (1) channel geometry including width, depth, bed forms and alignment, (2) the sorting process by which sand size decreases downstream, (3) scour phenomena, and (4) the features of sedimentary deposits. Further study of the mechanics of the formation, migration, elimination and effects of these large bars warrants detailed consideration, both in straight reaches and in bends. They must have a very significant effect on river behavior. Within the straight reaches these large bars develop in an alternate pattern -- one on one side of the channel, the next on the other side -- but further downstream, such that the main current meanders around them. These large bars move and are affected by changes in the geometry of the channel. When they reach a bend they are radically modified or destroyed by the secondary circulation, and within the bend a point bar is formed which is attached to the convex bank. These point bars have shapes dictated by the geometry of the bend, flow parameters, bank material and bed material load. In very wide channels with large width to depth ratios, large bars may form within the channel away from the banks. In some cases these bars may be reinforced on their upstream point in such a way that with subsequent scour and changes in the geometry of the stream they become semi-permanent islands.

It may be possible to photograph the occurrence and movement of large bars and possibly dunes in rivers using new photogrammetric techniques (Ref. II-19). Many new films are available from "Special Sensitized Products Division, Eastman Kodak" which may be used in various ways to study channel relief and quality of water.

Principal Variables

The forms of bed roughness and the resistance to flow in alluvial channels are complicated by the large number of variables and by the interdependency, either real or apparent, of the variables. In addition, some variables may be altered or even determined by the flow, and changes in flow conditions may change the role of a dependent variable into that of an independent one. It is difficult, especially in field studies, to select the independent and dependent variables from the possible candidates.

In a recent study, Simons and Richardson give a comprehensive analysis of the variables affecting bed configuration and flow resistance, their dependency and independency, the conditions whereby a dependent variable may become an independent one, and which variables may be eliminated as a first approximation (Ref. II-16). A detailed discussion is beyond the scope of this paper, but it is appropriate to list the variables affecting the bed forms and to discuss, in general terms, the effect or importance of the different variables.

The variables that fix the form of bed roughness in alluvial channels of stable width are:

$$\text{form of bed roughness} = \phi\,(D,\ S,\ d, \sigma, \rho,\ g,\ w,\ S_{\hat{c}},\ f_{\hat{s}}) \qquad (1)$$

where $w = \phi\,(d,\ \rho_{\hat{s}}, \rho,\ g,\ S_{\hat{p}}, \mu)$

and in which

D = depth

S = slope of the energy grade line,

d = median diameter of the bed material,

σ = measure of the size distribution of the bed material,

ρ = density of the water-sediment mixture,

g = acceleration due to gravity,

w = fall velocity of the bed material,

$S_{\hat{c}}$ = shape factor of the cross section,

$f_{\hat{s}}$ = seepage force in the bed of the stream,

$\rho_{\hat{s}}$ = density of the sediment,

$S_{\hat{p}}$ = shape factor of the particles,

μ = apparent dynamic viscosity of the water-sediment mixture.

The velocity of flow and the concentration of bed material transport, which some investigators have suggested are variables, have been omitted from equation 1. Under the restraint of equilibrium flow, both are dependent variables.

The effects of fine sediment on the apparent viscosity of the water-sediment mixture are considered included in the viscosity term. Fine sediments do not appear to influence the turbulence structure or velocity distribution (Ref. II-11).

In a natural stream, the energy losses and non-uniformity of flow caused by the bends and non-uniformity of the banks can have an important influence on the sedimentary structures generated by the flow, and a shape factor for the reach should be included in equation 1. With straight flumes with parallel walls the shape factor of the reach is a constant and can be disregarded.

Both the shape of the reach and of the cross-section are partially responsible for the fact that two or more bed configurations can occur side by side in natural streams.

The seepage force (f_s) may be a significant variable. As Simons and Richardson pointed out, in alluvial channels there is usually either inflow or outflow from the channel through the banks and bed material that causes seepage forces in the bed (Ref. II-16). If there is inflow, the seepage force acts to reduce the effective weight of the sand; and consequently, the stability of the bed material is reduced. If there is outflow from a channel, the seepage force acts in the direction of gravity and increases the effective weight of the sand and stability of the bed material. As a direct result of changing the effective weight, the seepage forces can influence the form of bed roughness and the resistance to flow for a given channel slope, channel shape, bed material, and discharge. Also inflow of seepage water can wash lenses of sand from the bank, causing bank failure, and can affect the stability of bank protection works.

The relation of the variables in equation 1 to the bed configuration will be discussed in the following paragraphs. The variables will be considered individually to avoid masking the essential role of any one of them.

Depth and Slope: Slope enters as a factor in equation 1 primarily through its role in the shear stress (γDS) as the driving force responsible for particle motion, and, at steep slopes with a corresponding steep bottom slope, through its contribution to a gravity component of the submerged weight of the sediment in the direction of flow (Ref. II-1). With changes in shear stress, the sedimentary structures generated at the bed by the flow change. To develop a given form of bed configuration in alluvial channels that have the same bed material, the slope will be steep if depth is small, and the slope must be flat if the depth is large.

Depth also enters the shear stress. The combination of depth and slope in the shear term (γDS) and its relation to particle size has been investigated by Shields, Bagnold, Brooks and many others (Ref. II-1, 2, 15). However, depth is also important as a scale factor which limits the ultimate size of the bed features; it relates significantly to the related roughness; and perhaps it is a factor which influences the turbulent energy spectrum. In addition, depth plays an important role in the bed material transport (Ref. II-15).

In uniform natural channels, where slope is approximately constant, the same sequence of bed forms observed in the laboratory as a result of changing slope may result from changes in depth. Depth plays an important role as a scale factor. For example, the flow depth determines the height of the large alternate bars which form at high width-depth ratios, as well as the scale of any superimposed features. Another example of depth effect is given by Nordin, where for the Rio Grande and tributaries, the wave length of standing waves was roughly related to flow depth (Ref. II-12).

<u>Size of Bed Material</u>: The effects of the physical size of the bed material on the formation of the bed configurations are (1) its influence on the fall velocity which is a measure of the interaction of the fluid and the particle, (2) its effect as grain roughness, and (3) its influence on the turbulent structure and the velocity field of the flow.

The physical size of the bed material, as measured by fall diameter (Ref. II-4) or by sieve-diameter, is a primary factor in determining fall velocity, although it is only important as it is related to the other variables in equation 1. Fall diameter has the advantage over the sieve-diameter of including the role of the shape factor and density of the particle as variables. Knowing only the fall diameter, the fall velocity of the particle in any fluid at any temperature can be computed; whereas, with the sieve diameter, knowledge of the shape factor and density of the particle are also needed.

The physical size of the bed material is the dominant factor in determining the friction factor for the plane bed condition and for antidunes when they are not actively breaking.

With dunes, the physical size of the bed material also has an effect on resistance to flow. The flow of fluid over the backs of the dunes is affected by the grain roughness, although the dissipation of energy by the form roughness of the dunes is the major factor. The form of the dunes is related to the fall velocity of the bed material.

Fall Velocity: Fall velocity is one of the primary variables that determines the interaction between the bed material and the fluid. For a given depth and slope, it determines the bed form that will occur and its characteristics. For example, the dunes are not only shorter in length, but they are also much more angular when the fall velocity of the bed material is relatively large. An increase in fall velocity requires an increase in the shear stress for the change from ripples to dunes or the change from dunes to plane bed.

With dunes, the smaller the median fall velocity of the bed material, the less angular they are and the larger their spacing for similar depths of flow. Also, the smaller the median fall velocity, the smaller the range of shear stress or of stream power within which a plane bed occurs.

For bed material with median fall velocity equal to or larger than 0.2 ft/sec, the range of stream power within which dunes occur is large and the range of stream power within which plane bed occurs is small. In fact, whether or not a plane bed occurs at all between dunes and standing waves is intimately related to fall velocity and depth. With shallow depths and bed material with relatively large fall velocity, the bed configuration may change from transition to standing waves without the development of plane bed.

The changes in bed form discussed in the preceding paragraphs were primarily caused by changes in the fall velocity (Ref. II-16). Viscosity and hence fall velocity were varied by changing the temperature or the concentration of fine sediment of the water-sediment mixture. With a change in fall velocity, other factors constant, the bed configuration and resistance to flow changed (Ref. II-9, 18). In the extreme case, by decreasing fall velocity, a dune bed could be changed to a plane bed, or a plane bed into antidune flow.

In addition to the flume experiments, observations on natural streams have verified that the bed configuration and resistance to flow will change with changes in fall velocity when the discharge and bed material are constant (Ref. II-8). For example, the Loup River near Dunning, Nebraska, has dunes as the bed roughness in the summer when the stream fluid is warm and less viscous, but the bed is essentially plane during the cold winter months. Similarly, data collected by Fahnestock (personal communication) on a stable reach of the Rio Grande show that, at similar discharges, when the water was cold the bed of the stream was plane, but when the water was warm the bed roughness was dunes.

Apparent Viscosity and Density: The effect of the various factors given in equation 1 on fall velocity are well known. However, the effects of fine sediment in suspension on fluid viscosity and fall velocity are less well known (Ref. II-18). The magnitude of the effect of fine sediment on viscosity is large and depends on the chemical makeup of the fine sediment. The changes in the fall velocity of the median fall diameter as a result of the changes in the viscosity and the fluid density can be noted in Figure II-2.

In addition to changing the viscosity, fine sediment suspended in water increases the mass density (ρ) and the specific weight (γ) of the mixture (Ref. II-18).

<u>Gradation of Bed Material:</u> The effect of gradation of the bed material on flow in alluvial channels, holding depth constant, where the gradation coefficient (σ) is

$$\sigma = 1/2 \quad \left(\frac{d_{50}}{d_{16}} + \frac{d_{84}}{d_{50}} \right) \qquad (2)$$

was reported by Daranandana and Simons and Richardson (Ref. II-6, 16). The numerical subscript denotes percent finer than; hence d_{50} is the median fall diameter of bed material.

In general, the bed features of the uniform sand were more angular than were the features of the graded sand.

Although quantitative relationships were not defined, the investigations definitely prove that there is a gradation effect on the bed forms and emphasize that natural river sands should be used in flume experiments if their results are to be extrapolated to the field.

<u>Prediction of Form of Bed Roughness in Straight Channels</u>

Knowledge of what the bed form is or may be under given fluid, flow and sediment conditions has important engineering application to channel control, to stable channel design, to determining the stage-discharge relations for natural channels, and to estimating the water and

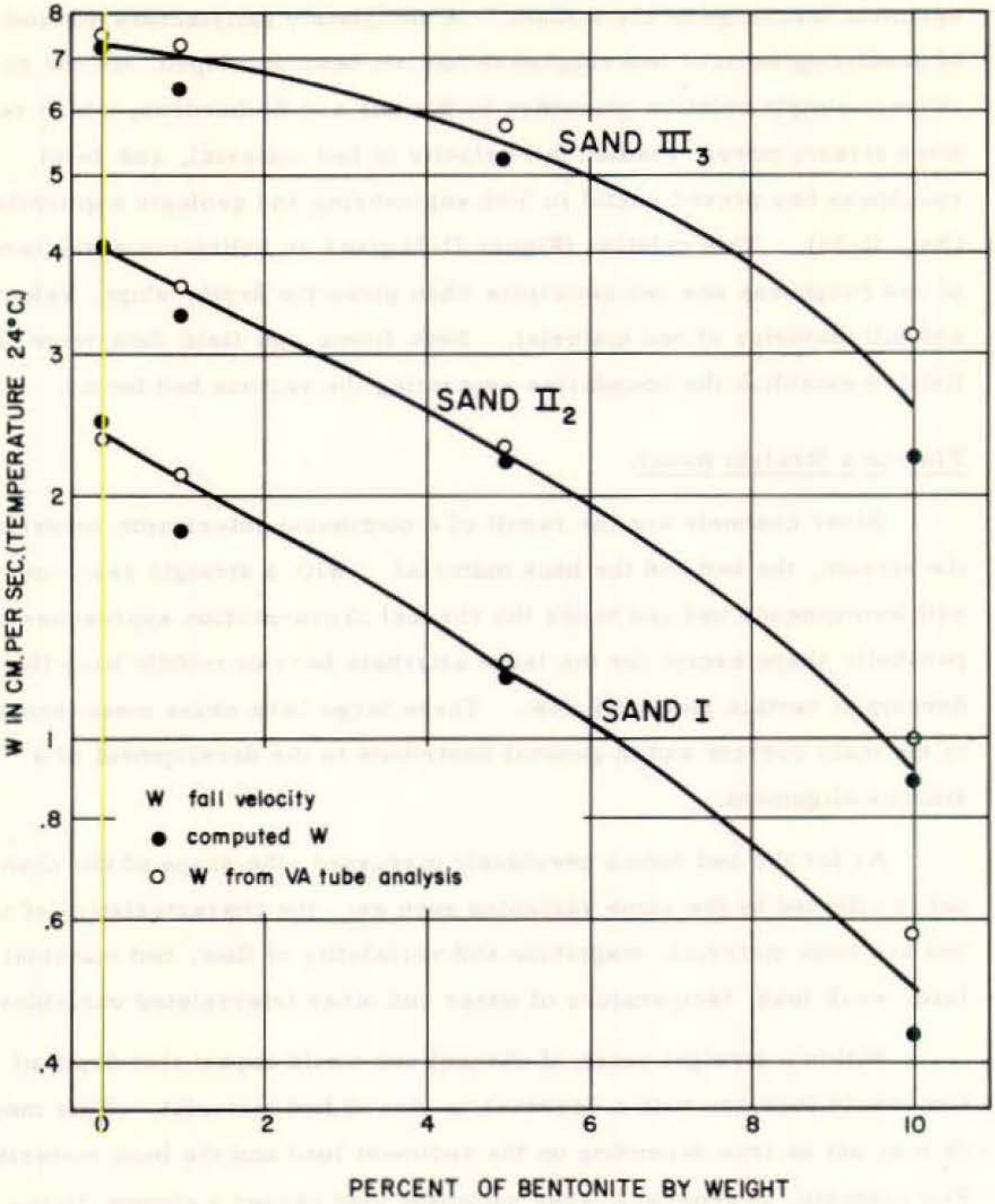


Figure II-2. VARIATION OF FALL VELOCITY WITH PERCENT OF BENTONITE IN WATER.

sediment discharge of the stream. A completely satisfactory method of predicting form of bed roughness has not been developed, but the relatively simple relation presented by Simons and Richardson, which relates stream power, median fall velocity of bed material, and form roughness has proved useful in both engineering and geologic applications (Ref. II-16). This relation (Figure II-3) gives an indication of the form of bed roughness one can anticipate when given the depth, slope, velocity, and fall diameter of bed material. Both flume and field data were utilized to establish the boundaries separating the various bed forms.

Flow in a Straight Reach

River channels are the result of a continuous interaction between the stream, the bed and the bank material. With a straight reach of river with homogeneous bed and banks the channel cross-section approaches a parabolic shape except for the large alternate bars or middle bars that develop at certain stages of flow. These large bars cause meandering of the main current and in general contribute to the development of a sinuous alignment.

As for the bed forms previously discussed, the shape of the channel is affected by the same variables such as: the characteristics of the bed and bank material, magnitude and variability of flow, bed material load, wash load, temperature of water and other interrelated variables.

Within a straight reach of channel one would expect that depth of flow would increase with a decrease in size of bed material. This may or may not be true depending on the sediment load and the bank material. For example, in general a large sediment load causes a channel to increase its width, but with semi-rigid or rigid banks the channel loses some of its freedom to adjust.

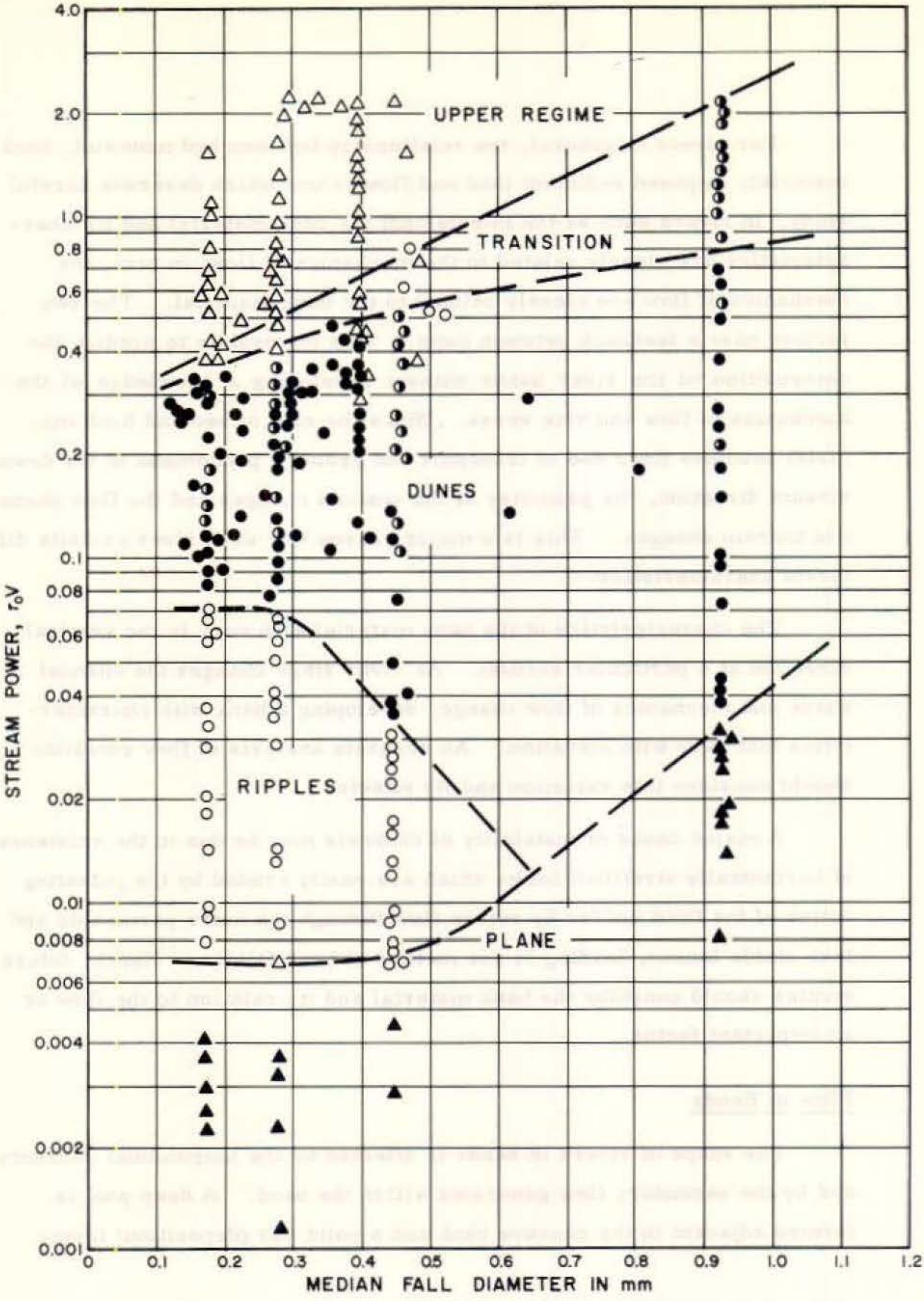


Figure II-3. RELATION OF STREAM POWER AND MEDIAN FALL DIAMETER TO FORM OF BED ROUGHNESS

For rivers in general, the relationship between bed material, bank material, imposed sediment load and flow is one which deserves careful study. In rivers such as the Mississippi the bank material and its characteristics are closely related to the mechanics of flow; in turn, the mechanics of flow are closely related to the bank material. The two factors have a feedback between them. It is impossible to predict the composition of the river banks without developing a knowledge of the mechanics of flow and vice versa. Since the size of bed and bank material becomes finer due to transport and geologic phenomena in the downstream direction, the geometry of the channel changes and the flow phenomena therein changes. This is a major reason why each river exhibits different characteristics.

The characteristics of the bank material also vary in the vertical direction at a particular section. As river stage changes the channel shape and mechanics of flow change, developing a bank with characteristics that very with elevation. An accurate analysis of flow condition should consider this variation and its effects.

A major cause of instability of channels may be due to the existence of horizontally stratified banks which are easily eroded by the pulsating action of the fluid and/or by return flow through the more permeable and less stable lenses, leading to one method of bank failure. Hence, future studies should consider the bank material and its relation to the flow as an important factor.

Flow in Bends

The shape of rivers in bends is affected by the longitudinal currents and by the secondary flow generated within the bend. A deep pool is formed adjacent to the concave bank and a point bar (deposition) forms at the convex bank. The channel is no longer somewhat parabolic. It is now nonsymmetrical, resembling a triangle as illustrated in Figure II-4.

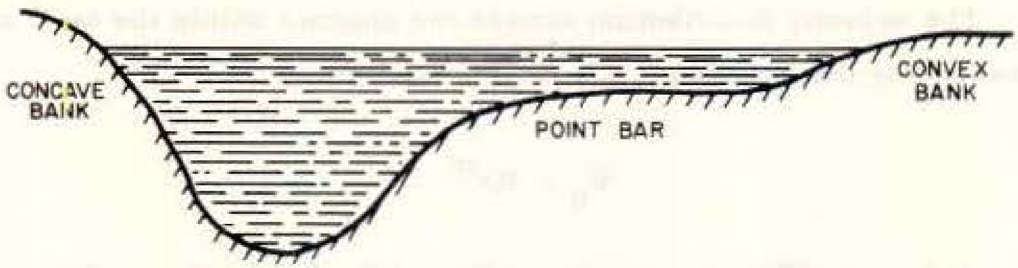


Figure II-4. TYPICAL SECTION ACROSS A RIVER BEND.

The water is superelevated at the concave bank causing a strong secondary flow which transports some of the bed material to the point bar or on downstream. The velocity distribution set up on the bend has a greater eroding capacity than within the straight reaches leading to a deeper than normal channel. The shorter the radius of the bend the deeper we expect the channel to be, but this requires qualification. The bend acts like a constriction; its effective area is less than its actual area. This causes an increase in depth in the upstream approach to the bend and an acceleration of flow in the bend. The smaller the radius of the bend the larger the backwater upstream and the greater the acceleration through the bend causing relatively large depths. Also, the smaller the radius of curvature of the bend the larger the superelevation of the water surface transverse to the flow and the stronger the secondary flow which also contributes to greater depth in the bend.

The added upstream increment of depth depends on the radius of bend, length of bend, geometry of the cross-section and the bed roughness generated in the bend by the flow. Research is needed to make possible the calculation of this increment of depth; however, useful results have been obtained based upon a field study of bends in lined canals at Colorado State University.

The velocity distribution across the channel within the bend can be expressed by the relation

$$V_0 = Kr^m \tag{3}$$

where K is a coefficient, r is the radius of the bend and m relates to degree to which a free or forced vortex is approached. When $m = -1$ the equation is the one for a free vortex and a frictionless fluid. When $m = +1$ the equation is that of the forced vortex. Actually, according to Einstein and Harder, the value of m is variable and may be as large as 4, depending on the characteristics of the bend (Ref. II-7). It can be concluded that no single mathematical formula has been derived which describes the distribution of velocities in the horizontal, normal to the banks in the bend of an open channel.

The superelevation within the bend is caused by the centrifugal force. The total superelevation between outer and inner banks can be derived by application of Newton's law. Several equations have been presented. One is based on the assumption that depth of flow upstream of the bend is equal to the average depth in the bend. This is applicable where high velocities occur near the outer bank of the channel (a forced vortex approximates the flow pattern across the channel). A relation has been developed for bends where the maximum velocity is close to the centerline of the channel and the flow pattern inward and outward from the centerline can be represented as forced and free vortices respectively.

For alluvial streams the superelevation will be predicted with adequate accuracy with the following relationship:

$$\Delta Z = V^2 b /_{rg} \tag{4}$$

where V is the flow velocity, b is the width of channel, r is the radius of the bend and g is the gravitational acceleration.

The various studies of bends have verified that the longitudinal velocity distribution changes with distance, cross-section, flow variables, fluid variables and fluid characteristics. For some bends the secondary circulation is of sufficient strength to completely destroy the normal distribution of the velocity in the vertical. With these changes which occur within a bend various forms of bed roughness can be generated, some of sufficient amplitude to interfere with navigation. When a bend is not properly proportioned the formation of large dunes may be triggered by such minor changes as relatively large variations in water temperature or changes in the concentration of wash load. For example, D. C. Bondurant (Corps of Engineers, Omaha) cited his experience with a bend on the Missouri River in which, for a particular discharge, the main channel would be adequate for navigation one day but would be partially plugged by large dunes which interfered with navigation at other times. In general, a detailed physical analysis of the processes of the interaction of the stream and the river bed is necessary to establish laws which determine changes in channel shape with distance, form of bed roughness, relief of water surface and channel stability. This involves the many significant, interrelated variables of equation 1.

Summarizing the existing knowledge of flow in bends, the rectilinear stream lines of the flow in a uniform straight reach are subjected to major changes as they enter the bend. These streamlines are given a general rotational movement, a secondary or transverse circulation, by the centrifugal force. The transverse and longitudinal components of flow combine to give the flow in the bend a spiraling motion. The water surface is superelevated at the concave bank. This produces a velocity field adjacent to the boundary toward the convex bank carrying bed material with it, some of which deposits and forms a point bar attached to the convex bank. The equations used to compute this superelevation were discussed earlier and are quite accurate.

Bottom currents of the river at curves undermine the outside bank and transport bed material toward the inside bank where at least part of it contributes to the development of the point bar. With deepening at the concave bank, the main current moves closer to it. The longitudinal velocity increases, causing depth to increase unless conditions favorable to the formation of large dunes develop, as has been observed in some bends. As a bend develops in an erodible material the forces generating secondary circulation are reduced, and the characteristics of the bend are stabilized. The bend will then move slowly downstream without much change in slope except as dictated by changes encountered in the bank material and changes imposed by new velocity distributions which result from changing stage.

With an increase in stage the depth and longitudinal velocities increase over the width of channel, but not uniformly. The increase in velocity at the concave bank is relatively smaller than at the convex bank. In the extreme, differences in depth at the two sides of the channel may become very small giving the channel a more nearly rectangular shape. The main current (dynamic axis) shifts from the concave toward the convex bank, straightening the main flow. When bankfull stage is exceeded there are further changes in the hydraulic regime of the stream. The water above the main channel and over the flood plain exerts a strong influence on flow in the main channel causing large changes in the hydraulics of the stream. Given sufficient time, overbank flow will lead to large changes in the river channel inconsistent with preconceived stabilization works. This problem deserves detailed study.

With lower regime flow the longitudinal profile of the water surface around a bend flattens over the deep sections and steepens over the flat sections where the pool terminates in favor of the shoal. That is, the longitudinal water surface configuration is out of phase with the bed geometry. The reverse is true if upper regime flow exists, but this is unusual and of short duration in large rivers because of the high velocity, rapid rate of bank erosion and widening, which quickly puts the stream back in lower regime. Upper regime flow should be avoided, but it has been observed at cutoffs where the channel was given an excessive slope sufficient to cause upper regime flow until the channel readjusted.

<u>Prediction of the Behavior and Alignment of Rivers</u>

The prediction of channel behavior and alignment is affected by the many interrelated variables upon which they depend. Little has been done in this country to relate the various interrelated variables, through the mechanics of flow, to river behavior and form.

A review of past work by Altunin, Pechkurov, Lokhtin, Boussinesq, Fargue and many others, as well as recent progress, was reported by Rzhanitsyn (Ref. II-14). The information which he presents may be used as a guide to further study and to assist with the determination of the probable channel alignment, relief of the river bed for given surface contours, the establishment of a navigable channel within the river and to help determine the position of rectifying structures for channel control.

Progress has resulted from an analysis of both laboratory and field data. This analysis has lead to the development of general mathematical parameters, figures and subclassifications of streams that can be used with some degree of success.

The parameters and relations between parameters will be discussed briefly.

Types of river bends are broadly classified into:

1. Free bends -- both banks consist of alluvial flood plain material which is easily eroded (meanders).

2. Limited bends -- the banks are of consolidated more stable virgin material which limits the degree of lateral erosion (entrenched meanders).

3. Forced bends -- the stream impinges on a bank of virgin materials at 60 to 100 degrees which forces a sharp unnatural bend to form.

In all cases the characteristics of the bank material determine to some extent the radius of curvature of the channel.

<u>Radius of Bends:</u> Average values of the ratios of the radii of curvature to width of stream at bankfull stage are:

1.	Free bends	4.5 - 5
2.	Limited bends	7 - 8
3.	Forced bends	2.5 - 3

The next important point is the variation of depth along the bend. In types 1 and 2 the depth increases with distance and reaches a maximum downstream of the apex of the bend. In type 3 depth increases rapidly at the beginning of the bend, reaching a maximum about one-third through the bend where there is concentrated deep scour.

Based upon flume and field data, numerous equations have been presented which relate radius of curvature to depth, slope, discharge and velocity. The following two formulas have been used and are accurate within about 40 percent.

$$r = 40 \sqrt{A} \tag{5}$$

$$r = \frac{0.004}{S} \sqrt{Q} \tag{6}$$

where

r = radius of curvature in meters

A = area of stream cross-section in square meters

Q = discharge in cubic meters per second

The major defect in these equations is that they do not adequately consider the individual elements of the channel, their interrelation, the length of the bend and the variation in the mechanics of flow along the bend. To improve these relations, Rzhanitsyn used a more theoretical approach and the basic equations of hydraulics to suggest that

$$r = \frac{\sqrt{K_o} \quad Q}{U_* \sqrt{A} \quad (\pi - \theta_0)} \tag{7}$$

where the new terms are defined as:

U_* = the shear velocity, $\sqrt{g \quad DS}$

θ_o = external angle of the bend in radians

$K_o = \Delta D_o / \Delta D_n = \ell \, rg/bC^2D$

and

D_o = longitudinal drop in water level

D_n = transverse drop in water level

ℓ = length of bend, $2\pi r\ (1 - \theta_o/180)$

b = channel width

D = channel depth

C = Chezy resistance coefficient, $V = C \sqrt{g\ RS}$ which varies with bed roughness.

The value of K_o depends upon the geometric dimensions of the cross section, the radius of the bend and the roughness coefficient; or in more obscure form, K_o depends on the result of the interaction between the stream and the river bed and banks. This parameter has different values for the different types of bends. Working with an average K_o value for a particular type of bend and equation 7, one can estimate a suitable radius of bend if the other variables in the equations are known.

Type of Bend	K_o
Free bend	3.5 - 5
Limited bend	18 - 22
Forced bend	1.2 - 2.2

These values of K_o are based upon low flow conditions, but investigations indicate that K_o can be considered independent of water level.

Relative Depth of Bends: To define depths in a bend in a general way it is necessary to know: maximum depth, weighted mean depth and minimum depth (shoal) in the bend. These three depths enable one to approximate the longitudinal profile of the bed and water surface through the bend.

The relative depth is defined as the ratio of depth to width (D/b) for the low water channel and is related to the relative radius of curvature (r/b).

$$D/b = f(r/b) \qquad (8)$$

When $r/b > 10 - 12$ the value of D/b approaches a constant value depending on the size of stream. For large rivers $D/b = 0.01$ and for small rivers 0.04 where D is maximum low water depth in the bend. The value of D/b is dependent on suspended sediment load (mean annual turbidity). In general, the larger the mean annual suspended sediment load the larger the value of D/b. In fact, with large mean annual suspended sediment load the relation $D/b = f(r/b)$ is radically different, emphasizing the significance of sediment discharge.

By considering the relation between:

(1) the relative depth D/b and relative radius of curvature r/b.

(2) the relative depth D/b, the relative radius of curvature r/b and the stability index $Y = d\ b/D^2S$ of the river where d is particle size.

(3) the index of stability and the angles of the lines in the $D/b = f(r/b)$ relation.

(4) the average annual turbidity and the relative depth D/b.

It was possible for Rzhanitsyn to formulate a method of relating the changes in relative depth with changes in relative radius for rivers of different sizes, degrees of stability and stream order (large numbers indicating large rivers) with mean annual suspended sediment loads of 80 to 90 ppm by weight (Ref. II-14). The general relation is presented in Figure II-5. Using such curves the greatest depth of pool and how it changes with radius of bend can be approximated for various rivers with low mean annual suspended sediment load. To extend the results of this analysis to streams of larger turbidities, correction coefficients were suggested.

In summary, maximum and average depths in bends are determined by a complex interaction with the stability of the river, the sediment load, the size of the river as well as possible effects of variations in hydrologic factors and the morphology of the area not accounted for in the relations presented.

<u>Length of Pools</u>: The transverse currents generated by the bend begin to diminish as the flow leaves the bend. As the transverse circulation diminishes the pool terminates. An approximate hydraulic equation for estimating the length of the pool was presented by Rzhanitsyn as follows (Ref. II-14):

$$L = \eta \frac{C^2 D}{g} \left\{ 2 \left[1 - \left(\frac{b}{2r}\right)^2 \right] - 1 \right\} \tag{9}$$

where

b is the mean width

D is the mean depth

r is the radius of curvature

C is Chezy's coefficient

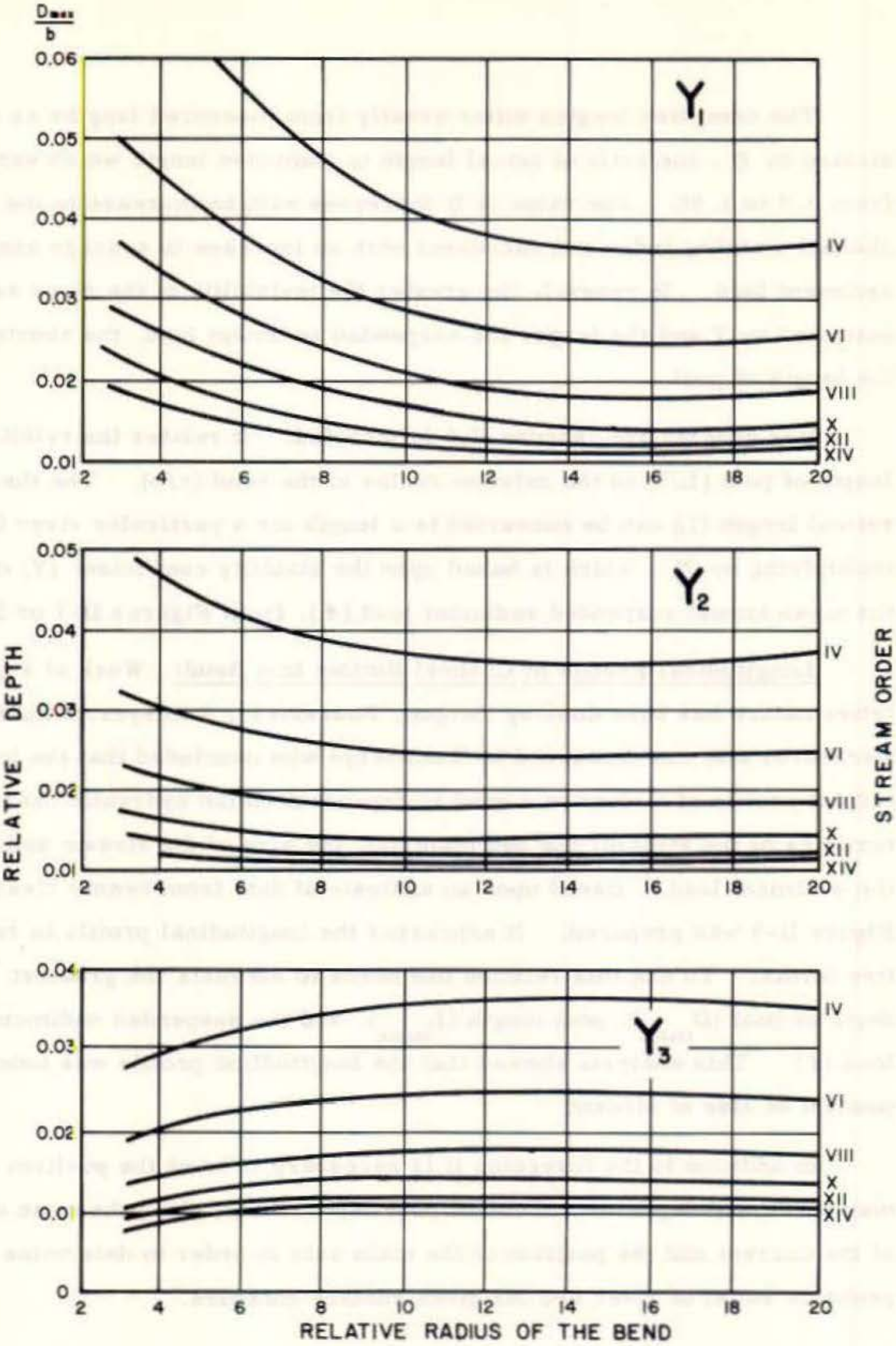


Figure II-5. CURVES OF $D_m/b = f(r/b; y)$ FOR NATURAL RIVERS WITH TURBIDITIES $\epsilon = 80\text{-}90\ gm/m^3$ (After N. A. Rzhanitsyn)

The computed lengths differ greatly from measured lengths as indicated by η, the ratio of actual length to computed length which varies from 0.4 to 1.90. The value of η increases with an increase in the channel stability index and decreases with an increase in average annual sediment load. In general, the greater the instability of the river as indicated by Y and the larger the suspended sediment load, the shorter the length of pool.

For general use, Figure II-6 is included. It relates the relative length of pool (L/b) to the relative radius of the bend (r/b). The theoretical length (L) can be converted to a length for a particular river by multiplying by η, which is based upon the stability coefficient (Y) or the mean annual suspended sediment load (ϵ), from Figures II-7 or II-8.

Longitudinal Profile of Channel Bottom in a Bend: Work of a qualitative nature has been done by Fargue, Boussinesq, Lameyer, Altunin and Pechkurov and was discussed by Rzhanitsyn who concluded that the longitudinal profile of a river in a bend is dependent on the hydraulic characteristics of the stream, the bed material, the size of the stream and on the sediment load. Based upon an analysis of data from twenty rivers, Figure II-9 was prepared. It expresses the longitudinal profile in relative terms. To use this relation one needs to estimate the greatest depth of pool (D_{max}), pool length (L_{max}), and the suspended sediment load (ϵ). This analysis showed that the longitudinal profile was independent of size of stream.

In addition to the foregoing it is necessary to know the position of maximum pool depth and minimum pool depth (shoal) along the main axis of the current and the position of the main axis in order to determine the probable relief of river bed for given surface contours.

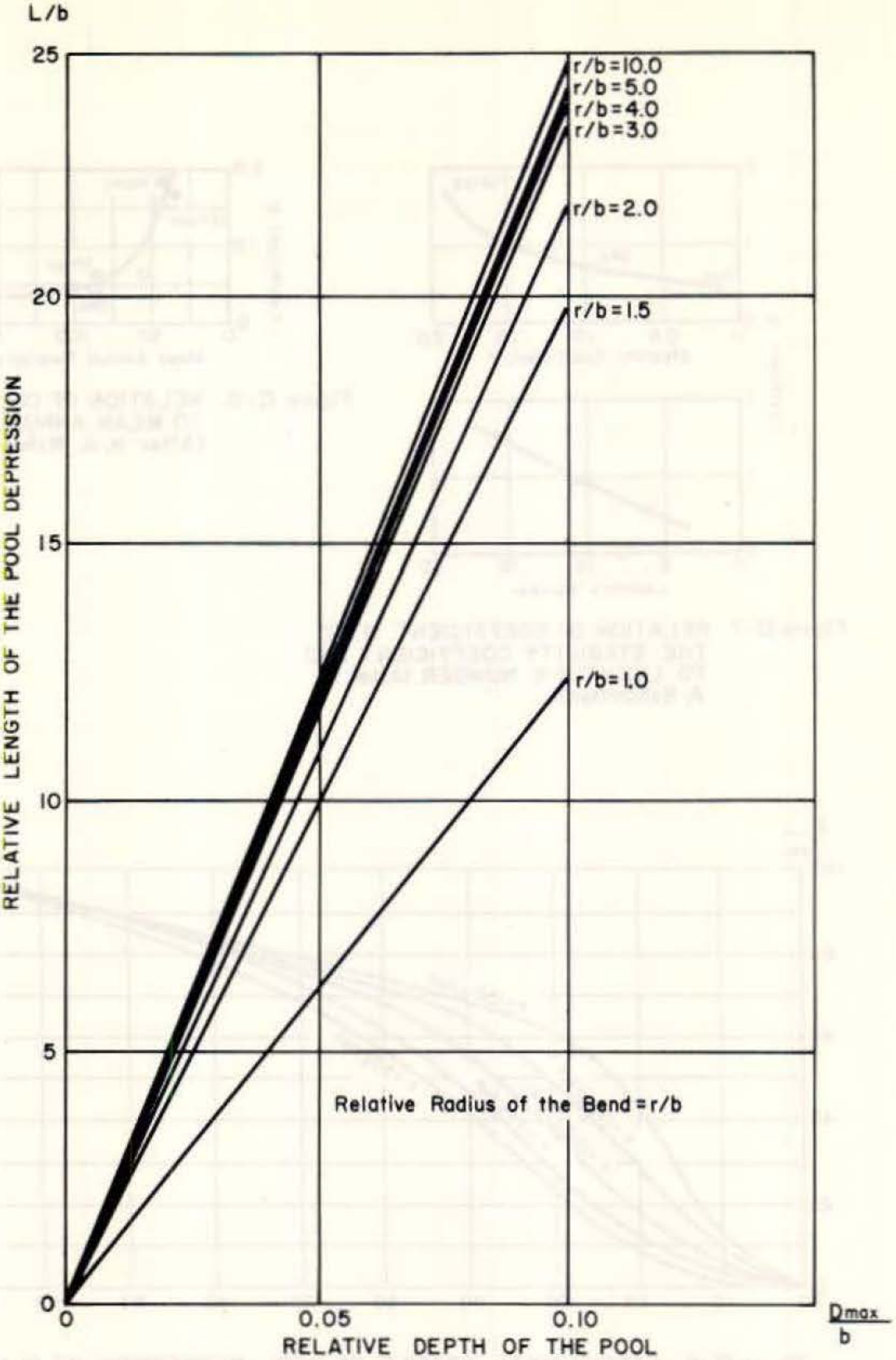


Figure II-6. RELATIVE LENGTH OF POOL DEPRESSION AS A FUNCTION OF MAXIMUM DEPTH AND OF RADIUS OF THE BEND. (After N. A. Rzhanitsyn).

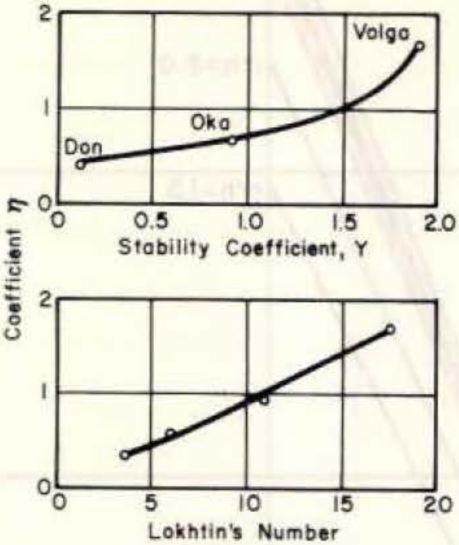


Figure II-7. RELATION OF COEFFICIENT η TO THE STABILITY COEFFICIENT AND TO LOKHTIN'S NUMBER. (After N. A. Rzhanitsyn).

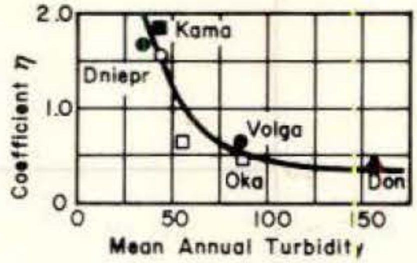


Figure II-8. RELATION OF COEFFICIENT η TO MEAN ANNUAL TURBIDITY. (After N. A. Rzhanitsyn).

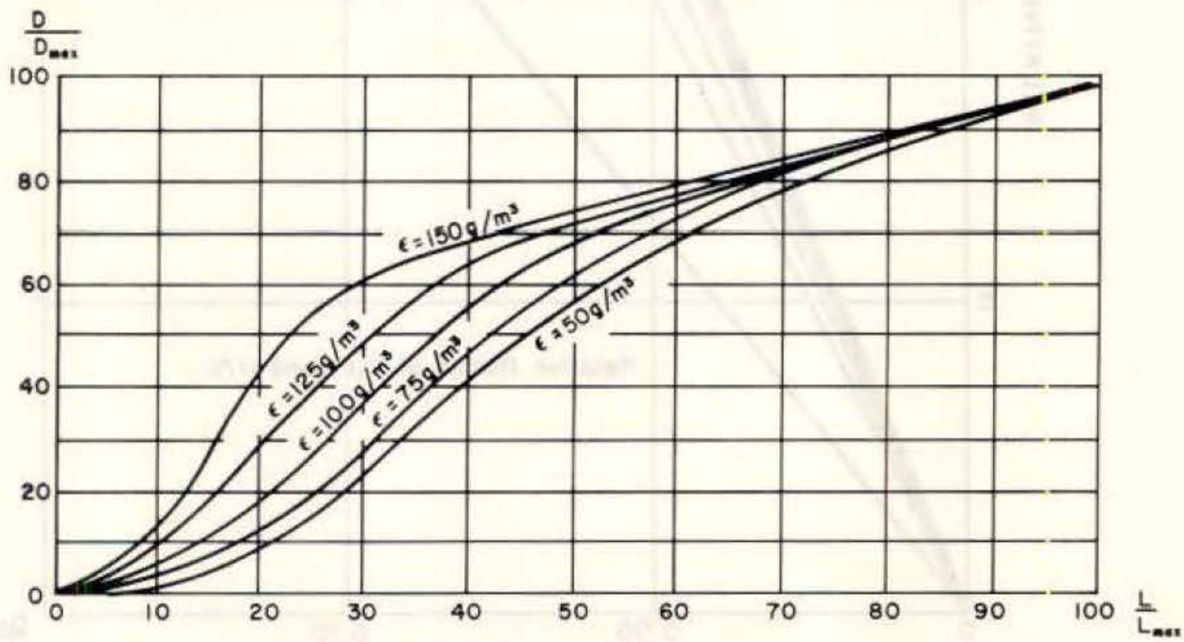


Figure II-9. LONGITUDINAL PROFILE OF POOL DEPRESSION OF PLAIN RIVERS —IN RELATIVE VALUES, COMBINED GRAPH. (After N. A. Rzhanitsyn)

The deepest part of the pool and the shallowest part (shoal) are located about one-fourth the length of pool (plus shoal) downstream from the points of maximum and minimum curvature for free and limited bends. The depth over the shoal can be obtained from Figure II-10. For forced bends the deepest part of the pool occurs at the section of maximum constriction during flood. Only approximate rules are available to estimate the position of the line of greatest depth around the bend, accordingly:

1. For free and limited bends in homogeneous non-cohesive material the greatest depth is located in the one-third to one-quarter width adjacent to the concave bank.

2. The greatest depth is within the middle third of the channel for forced bends.

3. The position of the line of greatest depth is similar for all sizes of streams.

4. In locating the line of maximum depth the influence of flow on the flood plain and the configuration of the valley which affect the flood flows should be taken into consideration.

Models

Some of the basic fundamentals of river behavior and control are well understood. However, a still better knowledge of the intermechanics and the physical laws governing flow in alluvial channels is of paramount importance to more economical and effective control of rivers. Most of the concepts presently utilized to analyze and solve river problems are based upon:

1. Idealized small scale flume studies.

2. Studies of straight reaches of canals and rivers.

3. Only limited studies of flow in bends which constitute the major portion of most river systems.

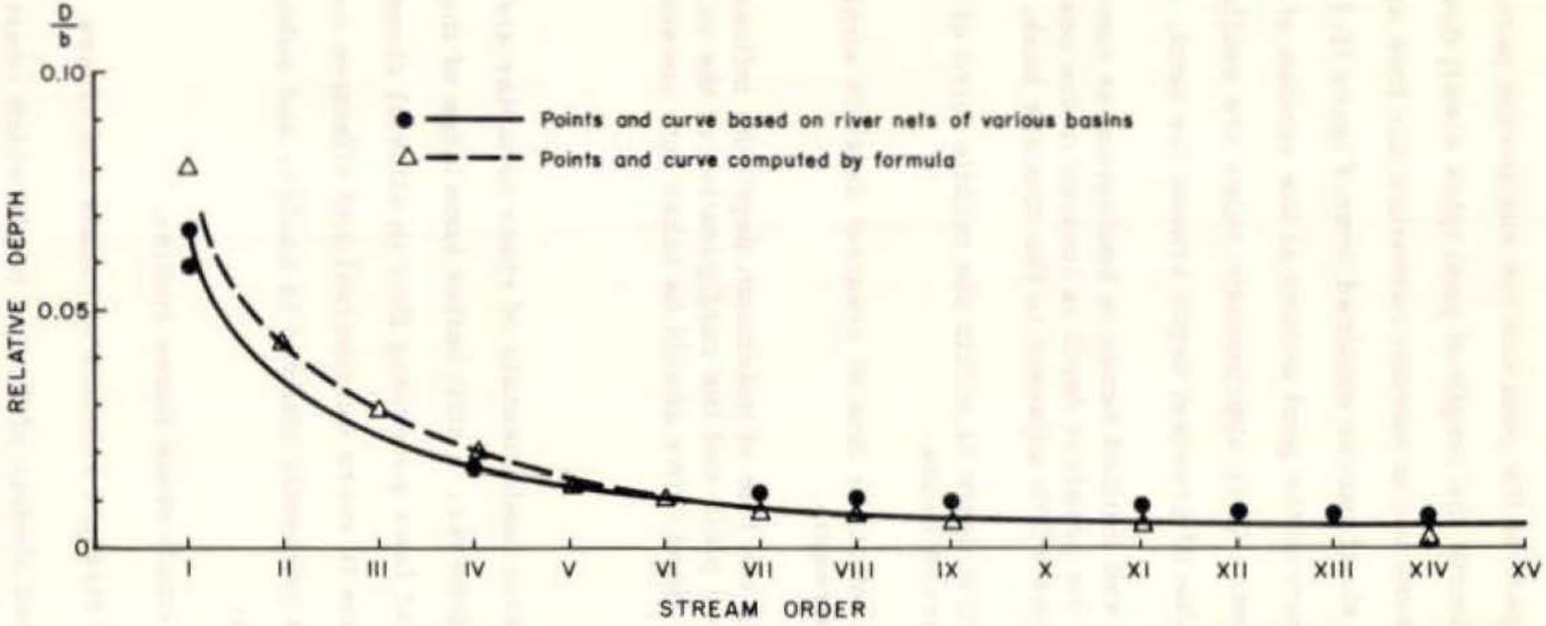


Figure II-10. RELATION OF RELATIVE DEPTH OF NATURAL STREAM TO THEIR STREAM ORDER.
(After N. A. Rzhanitsyn)

In essence, more emphasis must be placed upon studies of rivers in conjunction with laboratory studies. It is essential not to continue to bypass the rivers in favor of the less complicated idealized problems.

To help overcome this deficit of knowledge, river modeling can be and is often resorted to. However, because of the lack of a suitable theory the results of model studies are in themselves difficult to interpret and often misleading; but if properly used, model studies are very helpful.

Modeling of Natural Streams with Movable Bed Models: The use of hydraulic models, and their value, are unquestionably substantial in connection with design of hydraulic structures and hydraulic machinery, aircrafts, water crafts and the like. The principles of similitude are developed on the fundamental laws of motion of fluids and their relation to fixed or rigid bodies. Even though complete similitude is not possible in any hydraulic model of this group, at least sufficient time in history has passed in the use of models to assemble a practicable inventory of techniques to overcome the deficiencies. Successful methodology developed in fixed bed models enables accurate quantitative predictions from laboratory structures relative to the behavior of prototypes which are many times their size.

Unfortunately, however, such a similar positive statement cannot be made regarding the class of hydraulic models known as "movable bed models." Fundamentally this is because the laws of fluid motion relative to their movable boundaries and the hydrologic and morphologic features are not yet definite. Such phenomena as the interaction between the fluid and the moving boundary have received much detailed study, as discussed in the earlier part of this report, and deserve considerable additional study. The hydrologic problems of river mechanics are generating considerable interest in research, and the morphological aspects are being scrutinized from the viewpoints of detailed river sections and river systems as a whole. The list of needed research for better understanding of river behavior in both detail and general is long, and only a portion is indicated by the specific list following this section.

Model Scales: The basic requirements of all models is to achieve similarity of geometry including changes with time, similarity of motion and similarity of forces. The question of how to achieve these for a movable bed model from theoretical considerations and methodology, so that not only qualitative phenomena can be observed but some quantitative characteristics can be determined, is a complex problem that is not yet satisfactorily resolved in modeling practice. The theoretical considerations for modeling movable bed models discussed in this section are subject to revision as the fundamental relationships of river mechanics become better known.

Inasmuch as the modeling problem has been delineated to one involving water and sediment flow, the model similarity requirements can be grouped into two classes: (1) similarity of geometry, and (2) similarity of hydraulic characteristics. Geometric similarity involves such parameters as width, depth, slope, and sediment diameter. Hydraulic similarity includes discharge, velocity and turbulence, channel roughness, sediment transport (including a measure of interaction between the fluid and the moving boundary), flow duration and hydraulic time.

A relationship can be immediately established between the velocity scale and the depth or vertical scale, and where there is no vertical distortion, then to the horizontal scale. It therefore follows that either the velocity or linear scale may be chosen arbitrarily, and quite customarily the linear scale is selected. With the selection of a linear scale, relationships involving width, depth, and slope between model and prototype can be determined directly. Disregarding the friction criterion, the depth scale is an independent scale since it is a linear scale. Then if the horizontal scale is still an independent quantity, it follows that the slope scale can be determined.

The friction criterion definitely restricts the choice of either the vertical or horizontal scale, because the range of the friction factor "n" is bounded for both the prototype and for the movable bed model. This is contrary to the earlier assumption that both the longitudinal and vertical scales could be selected arbitrarily. In fact, if the ratio of resistances could be in some way determined from estimated or otherwise calculated values, then the order of distortion could be established. Moreover, it would appear from consideration of the first two criteria that sediment size and density for the model should not be selected arbitrarily but for their relationship to channel roughness.

Criteria of Channel Mobility: The dynamic similitude between model and prototype must hold, therefore, the corresponding forces related to the interaction of the flow and the movable bed must be similar. This implies that corresponding areas of the alluvial boundary must have similar bed motion, assuming a priori that velocities adhere to the Froude criterion. It is suggested that stream power (τ_0 V) encompasses, at least in part, the forces relating the interaction of the stream and its movable bed. From this expression a relation between median fall diameter of bed material in model and prototype can be developed.

Thus if the criteria of Froude, friction and channel mobility can be met, then geometric, kinematic and dynamic similitude requirements of model are satisfied. However, the criteria of friction and channel mobility are not often compatible, fundamentally because the relation ships between sediment diameter, channel bed forms, bed shear and channel roughness are infrequently unequally related. This fact does not invalidate the usefulness of movable bed models; for even though no scale distortion is best, some distortion in the various scale relationships are permissible. Fixed boundary models also have parallel problems of incompatibility in certain types of problems, which, as mentioned earlier, have been satisfactorily solved by appropriate methodology. The methodology of modeling is in a rapidly developing state at present for movable bed models. Both theoretical model relationships, stemming from continued research into the mechanics of water-sediment flow, and methodology of movable bed models deserves continuous and vigorous study. An additional item should be included in the list of research needs, and that is a study leading to the establishment of model scale relationships for movable bed models.

It must be concluded that a further effort to improve modeling techniques should be exerted. Also, to a greater extent, models should be used in conjunction with actual river work, not independent of it. That is, the model should be used to check the river and vice versa. By using larger scale models, light weight material such as crushed walnut shells, plastic particles (obtainable in angular or spherical form and with various specific weights), crushed bakelite and similar materials, it may be possible to study problems without using vertical distortion. This concept is at least worthy of further trial. However, it should be emphasized that the model, if used in conjunction with actual observations and work on the river, as a means of qualitatively developing control methods and related structures, should improve our modeling techniques, our understanding of river mechanics and lead to a more economical solution of river problems.

Additional Studies Needed

Considering all aspects of river control and stabilization, one of the most rewarding areas which should be pursued to improve methods and techniques is a coordinated interdisciplinary research and applied effort to develop a sounder theoretical basis for solving river problems. Some of the studies which should be encouraged follow.

1. Integrate existing knowledge of the mechanics of flow in open channels, river experience, geomorphology and soil mechanics to better cope with the design of bank stabilization and river control works. An interdisciplinary integrated effort is essential to further progress.

2. Study, using both flumes and rivers, the mechanics of flow in straight reaches paying particular attention to the relation between velocity distribution, geometry of cross-section, the bed configuration, formation of large bars and boundary shear stress.

3. New techniques are being developed using infra red and regular photographic techniques which indicates that there is a possibility of mapping channel alignment and relief including large bars and dunes. The development of this technique would enable the river engineer to study in detail the characteristics of a stream before and after modifications. This would help to rapidly develop the science of river mechanics and river control.

4. The stream channel forms as a result of complex interaction of the river, the bed and bank material and the imposed sediment load. The characteristics of both the river and its perimeter change with longitudinal distance

and with elevation at a cross-section. The complex feedback mechanism involved makes each river seem independent and unrelated to other rivers. It is necessary to understand this phenomena and to be able to take into consideration the variation of both flow phenomena and boundary materials in both vertical and horizontal directions as stage changes when designing bank stabilization works.

5. With the variation of flow phenomena with stage, the forces imposed on the bank stabilization also change. This is particularly true as conditions change from within channel flow to overbank flow. As overbank flow develops the upper layer of water exerts a large influence on the velocity distribution, boundary shear and sediment transport within the main channel. This new condition brings into play an altogether different set of forces on the bank stabilization works and hence should be understood and incorporated into the design. To improve knowledge of the effect of overbank flow it is essential to initiate a study of this phenomena.

6. The concept of using the geometry of bends requiring minimum maintenance to revise and stabilize other bends is an important one which is currently used. The river can be subdivided into reaches and within each reach the required modifications in geometry can be patterned after the bends causing the least problems. This procedure compensates for the effect of changes in bed and bank material, flow phenomena and channel geometry with distance and with further study could be made more effective.

7. Bank stabilization works should be designed to use the helpful characteristics of the flow and material in which the channel has formed. The river must be guided, not forced. Methods of stabilization must change with depth. No single procedure is fully effective over the complete range of normal, bank-full and overbank depths. Since it is impossible to have a completely flexible system of stabilization which changes with depth, it is essential to design for a characteristic flow about which there are minimum changes in shear stress on the perimeter of the channel. Designing for flows about which there is minimum variation leads to doing minimum work against the river and resulting in more economical works.

8. Statistical methods should be used to analyze and interpret data to establish possible relationships between the hydrograph, both present and past, and the dynamics of the river.

9. In river control there is a tendency to put the river in a straight jacket. In doing so the river is deprived of the sediment load derived from the banks of the channel. The river then picks up its sediment load from the bed causing degradation that can achieve large dimensions causing deepening of the channel and undermining and subsequent loss of bank stabilization works. This problem deserves serious study and consideration in developing river control.

10. As suggested in a recent ASCE panel discussion of stable channels, navigation channels must be designed to provide, with minimum flow, a section of adequate depth and width, generally within a larger section capable of containing periodic higher flows. This involves problems of bank stabilization, cross-section shape in relation to bend radii, cross-section shape in crossings between bends, flow depths in relation to bed roughness forms, and sediment transport. These items, in turn, require for their ultimate understanding much more knowledge than is currently available on matters of secondary circulation, flow distribution and boundary shear in bends, the causes and effects of variable roughness in a given cross-section, the relation of bed roughness to flow parameters and to sediment transport, the parameters involved in creating a stable bank revetment, the effects of various dimensions of dikes and other channel control structures on both the local and overall flow pattern, and the controls exerted by these flow patterns on the characteristics of the channel.

11. Better use could be made of models and rivers to solve river problems. Little can be gained by additional modeling of long reaches of river; but wide flumes in which, for example, a particular bend could be modeled quickly and qualitatively would assist with the design of bank protection work and help to illustrate the effect of varying stage and overbank flow on the river channel. The stream lines,

zones of separation and related surface flow phenomena can be studied quite simply in the rivers themselves by using floating lights and cameras to trace the currents.

PART II

Chapter 2

BANK AND CHANNEL STABILIZATION ON ALLUVIAL RIVERS

General

There are two basic types of river stabilization works -- direct and indirect. The design and installation of both types should be based on a thorough understanding of fluvial hydraulics and the physical and dynamic conditions in the particular reach of river under consideration. As stated in the previous chapter, probably the most effective and economical stabilization works are those which are designed to offer the least resistance to the natural flow patterns of the river. Those works which are installed without taking account of these considerations will often be completely lost or may aggravate more than benefit the situation. Therefore, to improve bank and channel stabilization techniques, it is necessary to expand our limited knowledge of fluvial hydraulics and study the various types of stabilization works that have been and are being used on alluvial rivers throughout the world.

Much of the information obtained by studying river control works and techniques has very little pertinence to the Lower Mississippi River. On the other hand, there are several types of stabilization works which are not now used on the Lower Mississippi but which might give insight into new methods. Most of these fall under the category of indirect stabilization methods. Some of the methods which may have significance are briefly discussed in this chapter.

Direct Protection

Direct protection methods (e.g., revetments) are the most prevalent type of bank protection on all rivers and have been used almost exclusively on the Lower Mississippi River. They prevent erosion by protecting the bank with a continuous erosion resistant cover, and have little effect on the river currents. The variations in the direct protection methods used on different alluvial rivers appear to be more a result of particular conditions of availability of materials, labor costs, and the characteristics of river, than any underlying theoretical concept. Even the philosophy that the best protection is fabricated from locally available materials such as willows and stone seems to be based to a great extent on the relative availability of such materials, and not entirely upon the economy and quality of the bank protection provided.

Most of Parts III and IV of this report deal with materials which might be used in directly protecting the banks on the Lower Mississippi. There being no significant distinctions behind the philosophies of using such protection on alluvial rivers, the various types of direct protection are not discussed in this chapter.

Indirect Protection

Indirect stabilization methods, to be effective, must influence the river currents. This can be accomplished in two ways; by deflecting the currents, or by retarding the currents. The currents can be deflected in such a way that they either do not impinge against the bank or in such a direction that the erosion is diminished; or the currents can be retarded to reduce their erosive capacity at critical locations. The usual function of both methods is to restrict the main river currents to a predetermined channel so that they are of sufficient velocity to keep it scoured.

They can also, however, be applied to bends for bank protection. In the following sections the two types of indirect structures, deflectors and retards, are discussed as they apply to both bank protection and channel stabilization.

Current Deflectors: Current deflecting structures have been used for many years on alluvial rivers for the purpose of stabilizing channels. Such structures have been installed to concentrate the river flow in one of several channels, to deflect the currents away from the bank, or to erode and maintain a navigable channel through a crossing. The use of deflecting structures to erode a channel through a crossing is perhaps most easily understood in terms of the explanation given by Leliavski (Ref. II-56) of the nature of the flow of water in rivers. He describes river currents as being composed of a series of "water-threads." When these threads diverge, the river loses its ability to transport the suspended sediment and deposition takes place; when they are parallel, there is no net deposition or scour; when they converge, there is scour. The water threads rarely run parallel to the banks, but rather tend to converge in bends, causing erosion, and diverge in crossings, causing deposition. Thus, to scour a channel through crossings it is necessary to deflect the currents in such a way that the water threads will converge.

Many different shapes and types of current deflecting structures have been installed in alluvial rivers. Most of these can be assigned to one of several general categories as follows:

1. High (Repelling) Groins
2. Submerged Groins and Sills
3. Bank Heads
4. Vanes
5. Training Dikes

The methods by which these structures are supposed to stabilize alluvial rivers and some examples of previous installations are given in the following paragraphs.

1. High (Repelling) Groins

Groins can either be impermeable, where they act primarily as deflectors, or permeable, where they act primarily as retards. They can also be either high, where their crest is above low water, or low (submerged), where their crest is continually below water. High impermeable groins (sometimes called repelling groins) are discussed below. Low groins are discussed in the following section, and permeable groins under current retards.

Impermeable high groins can be constructed in many different shapes. Some of the more common are:

1. Straight Spur Groin
2. Hockey Head Groin (with curve in upstream direction)
3. Inverted Hockey Head Groin (with curve in downstream direction)
4. Delaney Groin (or T-Headed Groin)
5. L-Head Groin (head pointing downstream)
6. Inverted L-Head Groins (head pointed upstream)
7. Hook Groins

Some of these shapes are more economical in certain locations than others. For instance, the Delaney Groin is economical in broad shallow rivers because it can protect one-half mile of river bank with a 300-foot long head. In narrow deep rivers, however, it would not be particularly economical because the majority of the construction would occur in deeper water.

Tests have indicated that some of the shapes are also more effective than others. The Hockey Head Groin sometimes tends to attract the current towards the bank rather than repel it, and thus the Inverted Hockey Head is often preferred. The Delaney Groin seems to be most effective when the greater part of the head is built on the upstream side. Similarly, the Inverted L-Head Groin appears to be more effective than the normal L-Head Groin.

The depths occurring in the Lower Mississippi River prevent the use of high groins for protecting most concave banks. However, by constructing them so that their heads are in appropriate alignment, they could be used in place of training dikes, as well as in confining the current through crossings. On the Middle Mississippi it has been found that, with the reduced silt load resulting from the construction of reservoirs and bank stabilization on the Missouri, impermeable groins are in many instances superior to permeable groins because (Ref. II-115, 136):

1. They are more effective in causing silt deposition.
2. They are less costly to maintain.
3. They have a longer life.
4. They are not subject to serious mechanical damage by floating objects because of their durability and their tendency to repel floating objects.

2. Submerged Groins and Sills

Submerged groins are impermeable deflecting structures which decline into the river, from the upper bank to the thalweg, with their crest approximately parallel to the transverse slope of the cross-section. They are often built on the concave banks where the depth is too great to allow the construction of a high groin, their purpose being to deflect the current away from the bank.

A sill is an impermeable submerged structure built across the thalweg with its crest approximately at a constant elevation. Their purpose is to deflect the currents away from the bottom and to constrict the depth, thus causing the river to widen. Often a sill is built concurrently with and as an extension of a submerged groin. Submerged groins were used on several American rivers during the last century (Ref. II-101) and some have recently been built on the Middle Mississippi (Ref. II-115, 136). They have also been experimented with extensively on the Rhine River (Ref. II-95, 118, 124) and have been used for a number of years on several Taiwan rivers (Ref. II-94, 95). Sills have recently been constructed on the Middle Mississippi and the Rhine, and have long been in use on the Rhone.

Submerged groins built of logs and filled with rock have been used extensively in Yugoslavia on the Morava River and others. These structures have a long life, give good protection and offer considerable resistance to flow.

The recent constructions of combination submerged groins and sills on the Middle Mississippi, though mostly less than four years old, are reported to be very effective. They are designed to constrict the flows at low river stages, and are built in concave banks when the depths exceed 40 feet. The top of the sill is constructed to an elevation of 12 feet below the low water of record.

However, groins can also aggravate rather than mitigate the erosion problems if they are not located properly. Van Ornum reports (Ref. II-101) that the early installations on the Lower Mississippi were only partially successful in protecting the banks because of the serious eddy currents produced between them. Near Helena, where the dikes were spaced at intervals of 400 feet "the slowly progressive recession of the bank was not only marked between the spear dikes, but at their shore ends the caving had occurred to such an extent that they no longer reached the bank even at low water." Similar experiences are reported for other installations.

On the other hand, Van Ornum also reports that combinations of submerged groins and sills have been very effective on some European rivers. On the early installations on the Rhone, for example, such a combination was used "whose efficiency in forming a regular and relatively permanent channel proved successful where all other known means had previously failed."

3. Bank Heads

Bank heads are impermeable deflecting structures constructed at various points along a bank. By creating a series of erosion resistant points they are supposed to confine the river to the established channel. They have been installed on several American rivers, rarely with much success. Friedkin (Ref. II-39) conducted some model tests to determine the effectiveness of bank heads, and his conclusion should suffice to discourage their use.

"The bank heads stabilized the channel in that the lateral development of bend was checked. However, the deep holes which developed at the protruding points indicated that they would be very difficult to hold. If the bankheads were held the tests indicated that they would cause a sharp rise in stage. Also, in navigable rivers, the sharp turns at the protruding points together with the turbulent flow, would create hazardous navigation conditions."

4. Vanes

The bandals which have been used for many years on some of the rivers in India, especially the Brakmaputra and Ganga (Ganges) (Ref. II-63, 92, 95), are a type of vane. They consist of rows of bamboo poles driven into the river bed which are crossbraced, supported with struts, and covered with a bamboo matting. They are inclined downstream at an angle of 30 degrees to 40 degrees, and their purpose is to deflect the low flows of the river into one channel in order to maintain an adequate depth.

Vanes were also experimented with or proposed for use on the Lower Mississippi River in the latter part of the 19th century (Ref. II-65). The purpose of these experiments was to find an inexpensive method of dredging adequate channels through crossings. In one case the vane was attached to the bottom of the boat; and in another, boats were temporarily sunk on the crossings in such a way that they would deflect the current into one channel. Some of these methods were reported to be successful in deepening the channel, but after they were removed the channel would rapidly fill up again. For this reason they were considered ineffective. It is now known that such deposition will usually occur in a dredged channel regardless of the dredging method used, unless the river currents have been permanently modified.

Other types of vanes which has been developed recently are the floating and submerged vanes developed by the Russians (Potapov Method) and largely perfected by the French at the National Hydraulics Laboratory at Chatou. These vanes have been installed on the Gorai River in East Pakistan and proposed for the Niger River in Africa (Ref. II-63). They are shaped like turbine blades and, for the surface deflectors, are spaced along a floating boom. They have been proposed for protecting concave banks, for stabilizing channels, for increasing the capacity of canal intakes, and for acting as silt ejectors. Their use in protecting the concave banks is based on the theory that the flow of the transverse currents down the concave bank is the primary cause of bank erosion because these currents impinge upon the individual bank particles in their least stable direction (i.e., down the slope) and then transport them away from the toe. The vanes therefore are installed so that they deflect the currents in a direction opposite to that of the normal transverse circulation.

All of these installations and experiments have indicated that vane deflectors can be effective in stabilizing channels (within certain limited depths); although their effectiveness in stabilizing banks is less certain. However, there are several considerations which discourage the use of such installations on rivers the size of the Mississippi. Some of these are:

1. The turbine blade vanes are quite expensive and their high cost would probably only be justified for protecting valuable structures.

2. Floating deflectors, to be effective, often must be of such a length and located in such a fashion that they seriously interfere with river traffic and are very susceptible to mechanical damage by boats and floating debris.

3. Experiments at Colorado State University have indicated that, to be effective, vanes have to have a depth of about half the depth of the river.

4. Vanes are usually effective when the currents are acting in only one particular direction at one particular stage. At other stages they could suffer serious damage or their action could aggravate the erosive tendencies of the river current.

5. Training Dikes

Training dikes (training walls) are structures, that run almost parallel to the banks, whose purpose is to confine and direct the river currents. Occasionally such dikes or walls are built in front of the concave banks and are thus a form of direct protection. However, of special interest to this study are those that are connected to the concave bank near its downstream end and gradually curve out into the river towards the crossing. The purpose of these dikes is to direct the current through the crossing in such a way that the "stream-threads" converge and an adequate channel is scoured and maintained.

Training dikes have been used on several European Rivers, such as the Elbe (Ref. II-95), the Po (Ref. II-95, 130, 131), the Rhone (Ref. II-95, 112), and the Rhine (Ref. II-78, 118), which have undergone extensive stabilization, and have been proposed for the Niger (Ref. II-63). The German rivers, excellent examples, have two parallel training dikes winding along with the river, containing the channel between the walls.

The proper design of training dikes can be based upon many of the theoretical concepts and graphs presented in the preceding chapter. These equations and graphs can be used as a guide to determine the radius of bend, the relative depths of bends, the length of pools and the longitudinal profile of the bed of the channel through the bend. The relations are to some degree consistent with the concepts of achieving river control in such a way that minimum work is done against the forces of the river and maximum economy of construction maintenance and operation results.

In fact, there is sufficient information available to:

1. Approximate the probable relief of a river channel for given surface contours.

2. Develop realignment plans for a given channel consistent with the foregoing principles of adequate design and minimum expense.

3. Determine the most effective location for rectifying structures such as training dikes to maintain desirable channel conditions.

The methods of applying the theory of Chapter 1 to the foregoing three problems are briefly summarized below.

To determine the probable relief of a river channel it is necessary to predetermine the order or magnitude of the river, make an estimate of the average annual suspended sediment load (either from records or from suitable sediment transport equations) and compute the stability index

$$Y = d\,b/_{D^2\ S}$$

With this information the length of pool can be estimated from the relation

$$L = \eta\ \frac{C^2\ D}{g}\ \left\{ 2 \left[1 - \left(\frac{b}{2r}\right)^2\right] - 1\right\}$$

The position of the shoals and greatest depth can be determined from the rules previously discussed which depend on type of bend and other factors. The position of greatest depth in the pool can be determined from Figure II-5 (page II-35). The depth at different positions in the pool can be obtained from Figure II-6 (page II-37), knowing channel width. The depth at the shoal can be obtained from Figure II-10 (page II-40). With this information the probable relief of the channel can be sketched.

Knowing the size of stream (stream order), the stability index (Y) and the relations cited in the previous section, the position of the main channel (fairway) in bends can be investigated. According to Rzhanitsyn the results obtained have been acceptable.

For example, consider the channel in Figure II-11.

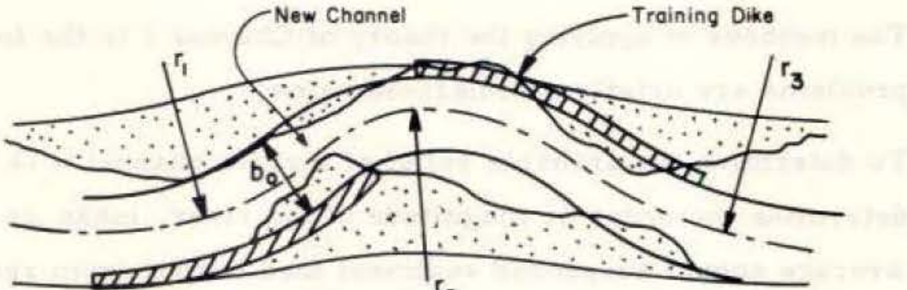


Figure II-11. POSITION OF NEW CHANNEL AND TRAINING DIKE. (After Rzhanitsyn 1963).

First determine the natural depth of straight channel with width b_o. The value of b_o can be computed using stability criteria and convential hydraulic formula such as the Chezy equation. Then select the acceptable low water depth. Using the appropriate curve from Figure II-5 (page II-35) and the selected channel width, determine the limiting maximum radius of the bend. That is, for D/b read r_1/b, and knowing b compute r_1. Next sketch the selected alignment of channel. The values of r_2 and r_3 may be equal to or smaller than r_1 without changing the width b. If radii larger than r_1 are used, this may lead to braiding at low flow and inadequate depth. After establishing the new alignment the probable relief can be determined and the design revised as necessary.

Another procedure which is widely used is to try and solve the problem by studying other reaches of the river. This procedure has merit and deserves further development.

Training walls are often used to create artificial bends in straight or slightly curved channels. The flow at the structure takes on a curved alignment equal to the radius of curvature of the structure. The further into the stream from the structure the less effect there will be on the alignment of flow. The wall causes a spiral flow of which the bottom layers are deflected toward mid-stream. This movement adjacent to the bed is similar to that caused by flow around a natural bend. One computational procedure which can be used to position training walls follows. The value of D/b is computed from given depth and width of desired channel, r/b is taken from Figure II-12 (page II-62) which relates D/b to r/b for natural bends. Then β is obtained from the relation of r/b to β in Figure II-12. The structure is then drawn in position extending into the river to the point where the angle of incedence is equal to β .

This procedure can be extended to other problems.

Current Retards: Current retarding structures have also been used for many years in stabilizing the channels of alluvial rivers. They are usually most effective in heavily silt laden rivers where, by reducing the current velocities, they induce deposition. The deposition is usually so rapid and of such a magnitude that the retarding structures are soon buried, which allows them to be constructed from such low cost materials as timber. Such timber pile dikes have been used for many years on European rivers and on the Lower and Middle Mississippi and the Missouri rivers, and thus need not be discussed in this report.

Fields of Kellner jetties have been used successfuly on several other rivers in the United States (Ref. II-29, 86, 140) for the same purpose, and other types of retarding structures (or hurdles) have been used on numerous rivers around the world. Although any of these structures might be

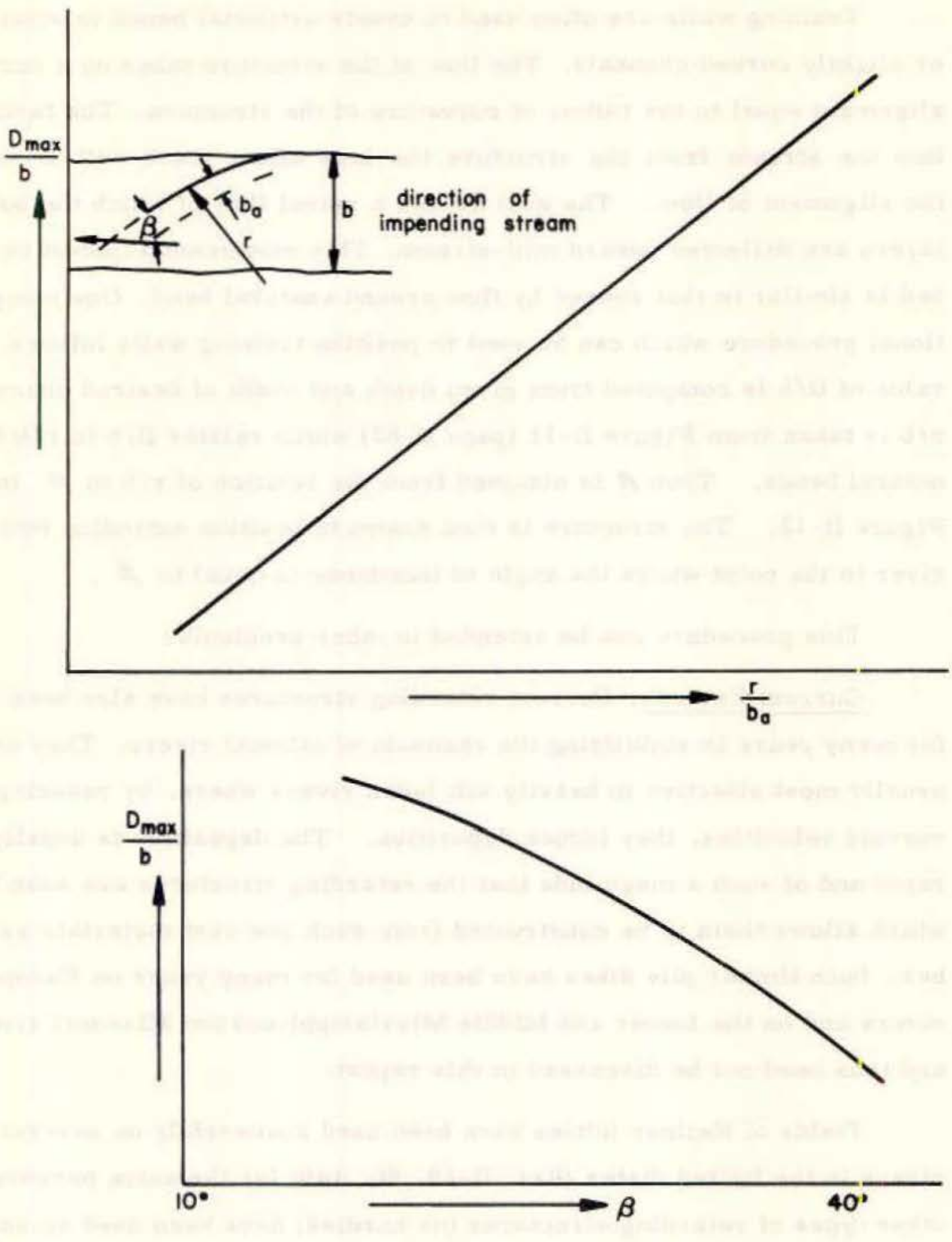


Figure II-12. SCHEMATIC OF $D_{max}/b = f(r/ba)$ AND $f(\beta)$ FROM EXPERIMENTAL DATA (After N. A. Rzhanitsyn)

used on the Lower Mississippi River, it is believed that because of the reduction in the silt load of the river resulting from the construction of dams on the Missouri, such retarding structures will become less effective in the future.

Current retarding structures have also been used in several forms for stabilizing and inducing deposition at the concave banks of rivers. Pervious fencing has been used on many small streams in California (Ref. II-27, 64), the Russian River (Ref. II-86, 134), and other streams in this country and abroad. Kellner type jacks have been used on Five Mile and Muddy Creeks (Ref. II-86, 122), the Russian River (Ref. II-86, 134) and the Canadian River (Ref. II-86, 107). Intermittent wire sausage on box shaped retards have been used on the Rio Grande River (Ref. II-86, 97), the Yellow River in China (Ref. II-93, 95), several rivers in Taiwan (Ref. II-94, 95), and the Red River (Ref. II-86, 129). Anchored trees have been used on the Russian River (Ref. II-134), on numerous rivers in Australia and New Zealand (Ref. II-83, 84, 85, 113), and on the Danube (Ref. II-78, personal communication).

Many of these installations were only considered temporary, and after the desired deposition had occurred, the bank would be permanently stabilized with some sort of revetment. This is called the dynamic approach. However, some of the structures have had a more or less permanent effect on the stability of the banks. Perhaps an adaptation of this type of structure might be used in stabilizing the banks of the Lower Mississippi River. In particular, it is possible that a type of structure called (in this report) a submerged retard, might adequately stabilize the river banks at a significantly lower cost than any revetment. This procedure requires continued surveillance so necessary work can be completed at the optimum time to minimize expense. No previous installation of such a structure on a large alluvial river is known.

The concept of the submerged retard has been deduced from data concerning the nature of scour in alluvial channels and from interpretations of the reasons behind the effectiveness of some other types of bank stabilization works.

Rock protection placed on the river banks has proven effective for small and medium size rivers. Quarry run rock which can be handled in a dump truck (two to three feet maximum diameter) is normally used. This type of bank protection is apparently effective for channels with depths less than about nine feet. In this case the bank roughness is sufficiently large to hold the thalweg and zone of maximum scour (or depth) of the stream far enough away from the bank to assure safety against high velocity currents and undercutting of the rock protection.

In contrast to the foregoing successful experience when similar protection has been tried in streams with depths in excess of about nine feet, the roughness of the rock was inadequate to hold the main current away from the protection works. Consequently the thalweg and zone of maximum scour moved in adjacent to the bank, undercut the rock, and caused the loss of protection works.

This suggests that success or failure depends on the relative roughness of the bank protection material where the relative roughness is similar to the relative roughness of pipes as defined by the Moody resistance diagram, Figure II-13, in which $e/_D$ is the relative roughness term, e is a measure of roughness height and D is the diameter of pipe. Using similar reasoning a relative roughness term equal to the effective height of bank roughness (assume equal to median diameter of rock protection when rock is used) divided by depth, $d/_D$, can be used. With the quarry run rock the protection has been successful when

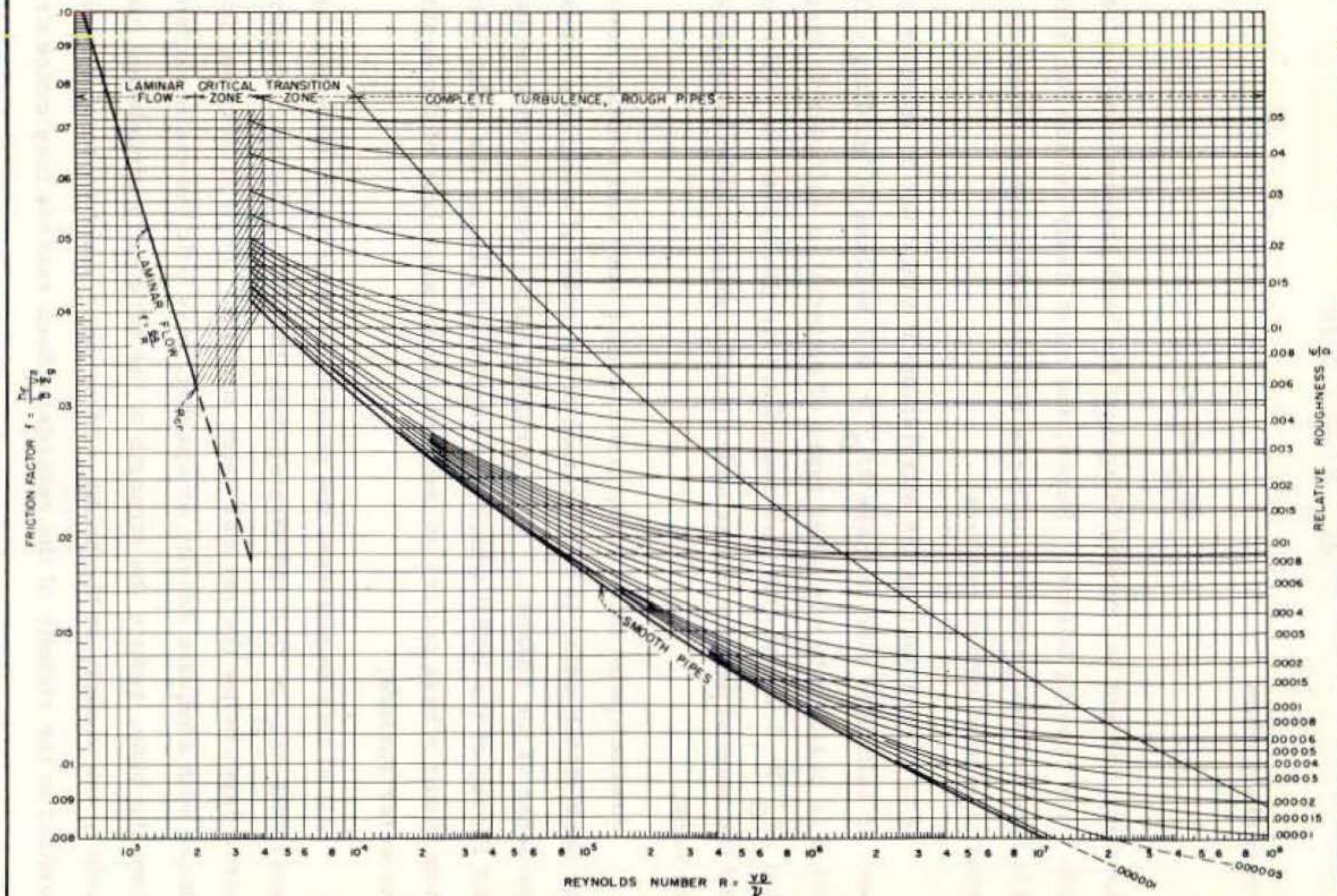


Figure II-13. MOODY DIAGRAM FOR FRICTION IN PIPES

$$d/_D > \frac{2}{10} = 0.20$$

conversely when

$$d/_D < 0.2$$

the main current has moved in against the bank undermining the protection work causing failure. Hence, the most likely reason for failure of this type of bank protector on large rivers is because of selecting too small a size of retard material.

For rivers such as the Mississippi it would not be feasible to use rock of sufficient size to make $d/_D > 0.2$ because of handling difficulty. But it is possible to use other types of material of sufficiently large size that $d/_D > 0.2$ and with this condition it is likely that the main current and the zone of deepest scour under the thalweg could be held away from the bank.

Another point of interest in discussing rock and similar materials for bank stabilization is the location of the point of maximum stress on the banks of the channel. In accordance with a study conducted by Olsen and Florey of the USBR which was reported by Lane, the point of maximum shear stress occurs on the bank about two-thirds of the depth below the water surface.

All of this emphasizes the possibility of using sufficiently large retards with $d/_D > 0.2$ and strategically located on the bank about two-thirds depth below the normal water surface. Such protection, properly placed with adequate density or spacing, may hold the main current away from the bank, reduce the strength of the secondary circulation (particularly in the vicinity of the bank), and perhaps promote some beneficial deposition in the vicinity of the retards. Such retards may concentrate the flow into the main channel, helping to scour and maintain a navigable channel, and reduce resistance to flow in the main channel by favorably modifying the bed roughness which reduces stage for a given discharge. Possible types of retards that may perform as indicated are:

1. Large roughness elements such as Kellner jetties with material attached to develop adequate resistance to flow to move the current off shore.

2. Large trees with bases anchored by chain to a sufficiently large concrete block to hold the line or lines of trees in proper position. This procedure has worked very well on the Danube River in Yugoslavia.

3. Shot or driven piling in the bank with attached materials to increase the resistance to flow in the vicinity of the bank.

Thus, the submerged retard would be a permeable structure placed at about two-thirds depth which would provide sufficient roughness to eliminate, in its vicinity, the erosion tendencies of the natural longitudinal and transverse currents. In addition, it could be constructed in such a way that it would also act somewhat as a deflector, counteracting the natural transverse currents and moving the main currents away from the bank.

It is believed that if the part of the bank at two-thirds depth were stabilized by such a method the rest of the bank might also be stabilized because:

(1) The transverse currents would not have a chance to reach the bottom of the bank where they tend to carry the sediment away from the toe causing deepening of the thalweg and undermining of the rest of the bank.

(2) The upper two-thirds of the bank would not be undermined and would be subjected to less erosive currents because of the elimination of the transverse currents flowing down the bank.

Although the degree of roughness required for such an installation to be effective cannot be predicted with great accuracy on the basis of existing knowledge, it can be estimated by analyzing some of the other installations referred to at the beginning of this section. The trees anchored in the concave bends of the Danube River reached from about the 15 to 25-foot depth to the water surface (personal communication) and thus could be said to have a height of approximately 15 to 35 feet -- about equal to river depth. These installations, however, caused rapid deposition, which would not be required of permanent bank stabilization structures.

PART II

Chapter 3

DESIGN CRITERIA FOR BANK PROTECTION

General

A preliminary requirement in pursuing this study was the establishment of criteria which could be used in evaluating the efficacy of proposed methods of bank protection for the Mississippi River on both the upper and lower banks. These criteria are generally applicable to bank protection or stabilization on any alluvial river, but take special account of the specific conditions experienced on the Mississippi, such as its great depth and high velocities. They have served a valuable function in allowing comparison between various materials and methods considered during this study. Their purpose has not been to absolutely define minimum requirements which must be completely satisfied by any particular type of bank protection, for it is doubtful that any economical bank protection could satisfy all requirements completely. Proposed methods of bank stabilization were generally evaluated by these criteria before any attempt was made to determine the economic feasibility of their utilization.

In this chapter each of the criteria applicable to riprap upper bank protection, monolithic upper bank protection, revetment type lower bank protection, and indirect bank protection is presented and briefly discussed. Discussions of the existing types of protection, their cost, and the degree to which they satisfy these criterion, are given in Part III(for the upper bank protection) and Part IV (for the lower bank protection).

Upper Bank Riprap Protection

In this study the category of riprap protection includes all protection formed by individual unconnected pieces of durable hard stone or other suitable material which, when placed in a sufficiently thick layer on the bank, are heavy enough to resist being moved by, and to protect the underlying bank from, the normal erosive forces of the river. The two properties of durability and erosion resistance mentioned in this definition are discussed below. Such protection also often requires a suitable filter beneath it to prevent loss of the foundation material. The requirements for such a filter are also discussed below.

Durability: The riprap material must be sufficiently durable to resist abrasion by flowing water with suspended silt and sand. It must retain its strength and soundness under the expected weathering conditions which include wetting and drying, occasional freezing and thawing, and sun baking. It also should resist crushing by the physical impact of boats and floating debris. Some of the standard specifications which could be used to evaluate the durability of proposed materials are:

AASHO T 104 (ASTM C-88)	- Soundness
ASTM C 131 or ASTM D-2	- Abrasion
ASTM D-3	- Toughness
AASHO T 85	- Freezing and Thawing
ASTM C 127	- Absorption

Erosion Resistance: The individual pieces as finally placed should be sufficiently heavy to prevent them from being moved by the anticipated stream velocities and wave wash. Velocities in the Mississippi River are said to be as high as 12 feet per second (Ref. B-31). Although the velocities near the banks are normally less, this value has been used as the criterion for design in order to take account of the possible existence of eddy currents. Waves resulting from wind and the passage of boats and barges reach heights of one to two feet. The bank upon which the riprap is to be placed will have been graded to a 3h:1v to 5h:1v slope.

The required weight for riprap to resist displacement by these conditions will depend upon the density of the material used. For this reason Table II-1 has been prepared to indicate the range allowable for the 50 percent size (weight in pounds) of riprap materials of different densities.

Table II-1

ALLOWABLE RANGE OF RIPRAP WEIGHT

Density (lb/ft^3)	Range of 50% Size Weight of Stone (lbs.)
170	20 - 30
160	25 - 38
150	32 - 48
140	42 - 61
130	56 - 81
120	78 - 115
110	120 - 175

In preparing the table, the required riprap sizes according to the TVA design procedures (Ref. II-87) have been modified to make them consonant with the existing specifications. The relationship between the required 50 percent size and stone density according to the TVA formulas is shown in Curve A, Figure II-14. The modified relationship is labelled Curve B. The dotted lines on each side of Curve B indicate the arbitrary limits which have been established for the range of 50 percent sizes in preparing Table II-1. These design procedures assume that the shape of, and interlocking between, individual pieces of riprap will be similar to that of quarried stone. If there is less interlocking, the individual pieces would probably have to be somewhat larger, and if there is more interlocking, they could be smaller.

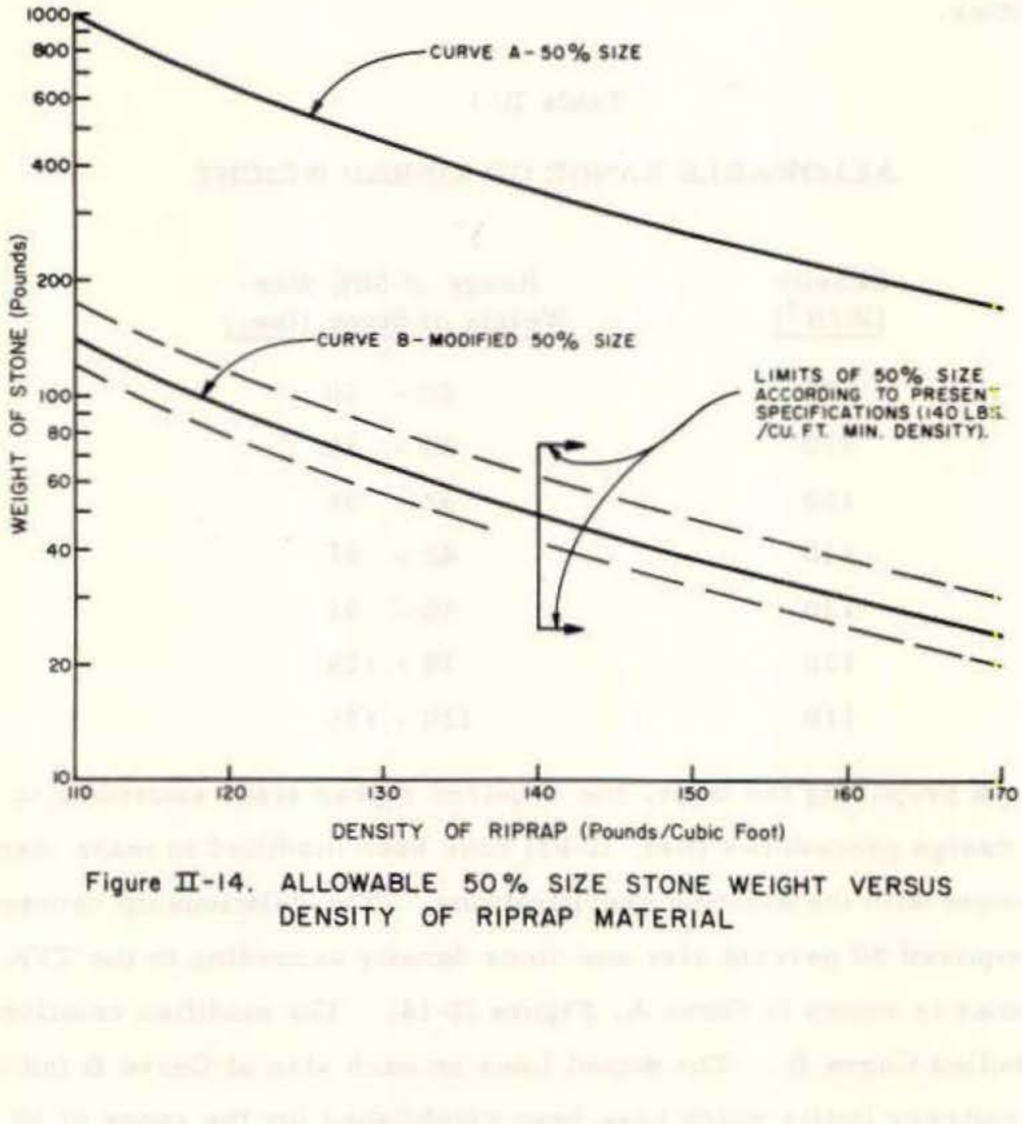


Figure II-14. ALLOWABLE 50% SIZE STONE WEIGHT VERSUS DENSITY OF RIPRAP MATERIAL

Filtering: Riprap protection is not normally capable of sufficiently filtering the foundation materials to prevent their removal by the river current or ground water seepage. For this reason a separate filter is usually required below such protection, especially when the foundation is composed of cohesionless materials. This filter can either be a graded aggregate according to the specifications established by the Waterways Experiment Station (Ref. II-103) or it can be a suitable fabric. The specifications for a graded aggregate filter are as follows:

$$\frac{D_{15}\text{ Filter}}{D_{85}\text{ Base}} < 5;\ 4 < \frac{D_{15}\text{ Filter}}{D_{15}\text{ Base}} < 20;\qquad \frac{D_{50}\text{ Filter}}{D_{50}\text{ Base}} < 25$$

The criteria for a sheet or fabric filter are given on page II-81. Such filters must also meet the criterion of durability either as given above (for aggregate filters) or in the following section (for cloth or perforated sheet filters).

Monolithic or Pavement Type Upper Bank Protection

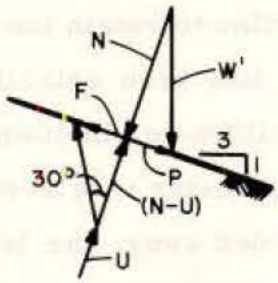
A monolithic or pavement type of protection can be used in place of riprap as upper bank protection. This category includes all protection methods or materials whose application is in the form of, or whose resistance to erosion mostly results from their being, a continuous integral protective covering of the bank materials. It includes, in addition to pavement protection placed on the surface, such methods as stabilizing the bank materials in place or covering them with a thin sheeting or fabric. Such methods require durability, strength, and flexibility. They must supply adequate filtering properties, but must also have sufficient permeability to allow ground water to seep back into the river. In addition, they must be suitable for placing on the prepared bank surfaces under anticipated climatic conditions. All of these criteria are discussed below.

Durability and Strength: The protection must meet the criteria listed under "Durability" on page II-70. It must also retain sufficient internal strength to allow it to act effectively for an expected life of not less than 25 years.

Flexibility: The material must have a sufficient flexibility that it can adjust to differential settlement of the foundation, irregular bank surfaces, and minor undercutting, without losing its strength. Or, if it is not flexible, it must break up into pieces which are of sufficient size and gradation that they will meet the requirements of erosion resistance and filtering properties outlined for riprap protection on page II-70 to II-73.

Permeability and Filtering Properties: The protection must either have sufficient inherent permeability or have suitable drainage features constructed through or beneath it to allow the drainage of groundwater from the banks into the river. The permeability or drainage features must be sufficient to allow drainage from the most permeable underlying deposit without excessive hydrostatic pressures. The most permeable deposits will probably be the fine to medium fine sands, which, according to Fisk (Ref. B-9), have a permeability varying from 10×10^{-4} to 50×10^{-4} centimeters per second. Thus the bank protection should be of this permeability or greater, or should have sufficient facilities to allow the drainage of 0.6 cubic centimeters of water per second per square foot of protected area without causing excessive uplift pressures. The allowable uplift pressures will depend upon the submerged weight of the protection. Calculations of allowable uplift pressures were made assuming a factor of safety of 1.0 against sliding down a 3:1 slope with a friction angle of

30 degrees and ignoring current forces. The method of calculation and the resulting relationship between allowable uplift pressures and submerged weight per square foot are given in Figure II-15.



W' = submerged weight per sq. ft. of slope

P = driving force of protection = $W'/\sqrt{10}$

F = frictional force of soil = $(N-U) \tan 30^{\circ}$

N = normal force = $(3/\sqrt{10})\ W'$

U = seepage uplift force per sq. ft. of slope

For a factor of safety of 1: $F - P = 0$

$$\text{Therefore:} \quad U = W' \left(3/\sqrt{10} - 1/\sqrt{10} \tan 30^{\circ}\right)$$

$$U = 0.4\ W' \text{ psf}$$

$$U = .95\ W' \text{ (head of water in cm)}$$

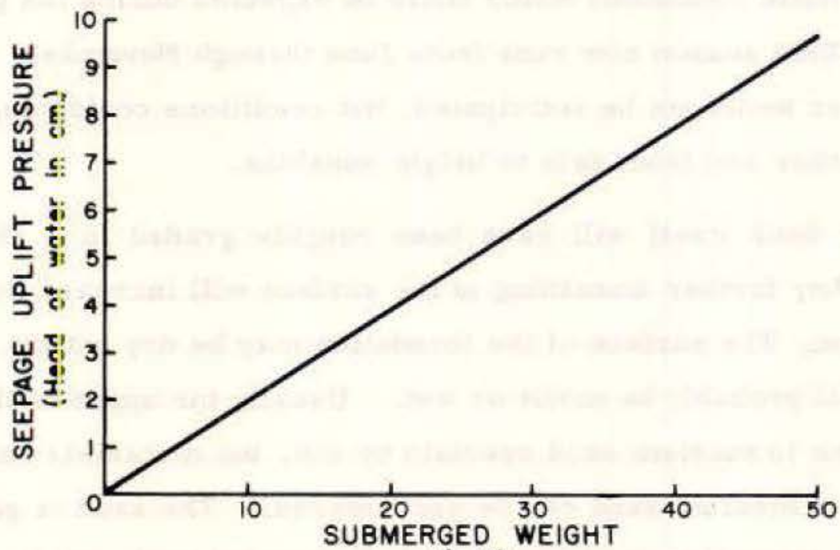


Figure II-15. ALLOWABLE UPLIFT PRESSURE VERSUS SUBMERGED WEIGHT OF PROTECTION.

If a filter blanket, discharging at the bottom of the upper bank, is to be used beneath the protection, it should have a sufficient capacity to allow the drainage of all the bank area above it.

Concurrently with this permeability, the protection or its drainage features must provide sufficient filtering properties to retain the fine foundation sands. For a sheet or fabric filter it has been established that the openings must not be larger than 0.2 millimeter in diameter. In establishing a maximum opening size of 0.2 millimeter it is assumed that after some of the finer surface grains are eroded away, the larger grains will bridge over the openings in the cloth and will prevent both any further erosion and clogging by finer grained deposits. This may not apply to a thin cloth which changing pressures and current velocity fluctuations can move readily.

Placement Conditions and Surface Preparation: Materials considered for protection of the upper bank should be suitable for placement during the normal climatic conditions which could be expected during the placement season. This season now runs from June through November. Thus freezing weather would not be anticipated, but conditions could range from cool to hot weather and from rain to bright sunshine.

The upper bank itself will have been roughly graded to a 3h:1v to 5h:1v slope. Any further smoothing of the surface will increase the cost of the protection. The surface of the foundation may be dry but the material below will probably be moist or wet. Usually the upper bank is composed of fine to medium sand overlain by silt, but materials ranging from fine clay to medium sand can be encountered. The sand is generally from loose to medium in density and both it and the silt can be expected to provide a loose uncompacted base for the protection.

Lower Bank Revetment Protection

Revetment protection is defined as any protection which directly protects the slope by providing a continuous, protective cover over it. It includes mattresses, sheeting, riprap, pavement, etc. Such protection is in contrast to indirect protection which will be discussed in the following section. Revetment protection of the lower bank requires many of the same properties as pavement protection of the upper bank. It must be durable and strong, and yet retain flexibility. It must be permeable and yet have sufficient filtering properties to retain the foundation materials. It must have sufficient weight or anchoring to secure it to the slope and must be suitable for placement under all normal conditions. These requirements are discussed below.

Durability: The material must be sufficiently durable to allow it to remain effective while continuously submerged and subjected to abrasion by suspended silt and sand and the impact of waterlogged trees and other debris being carried by the river. That portion of the revetment near the low water level will also be subjected to impact by barges and other boats. That portion of the revetment which will be continually submerged need not be especially resistant to sunlight, freezing and thawing, wetting and drying and extreme variation in heat, except to the degree that it will be subjected to these conditions before placement.

Strength: The material must have sufficient strength to allow placement and to maintain effectiveness under the service conditions listed under durability above and under all probable foundation conditions. The foundation will generally be cleared and graded to a slope of 3h:1v to 5h:1v to a distance of 30 to 50 feet below low water level. Below this depth the slopes may be as steep as 1.5h:1v (in clays) or approach the horizontal. There are apt to be tree stumps and other waterlogged debris imbedded in the bank surface, especially on the bottom surfaces.

Weller has stated (Ref. II-86) that "great compressive or tensile strength is not important. Only sufficient strength and bond to permit transportation, fabrication and launching without excessive breakage is required." The criterion which was used in establishing the required strength of the present articulated concrete mattresses was that they must be strong enough to be suspended in 100 feet of water while subject to a current of five feet per second (personal communication).

Although this criterion is the governing factor in determining the required strength of the present mattresses, it might not be applicable to some materials and placement methods. A mattress that is placed by unrolling it down the bank will require higher strength by durability and foundation conditions criteria than by the placement criterion governing the strength of present mattresses. The tear and puncture strengths of some materials will probably dictate the type of criterion that governs strength. Although specific strength requirements for each material and placement method are beyond the scope of this study, the variation of strength criterion from one material and placement method to another is considered in selecting the pertinent strength parameters for a material and the most feasible method of placing it.

Flexibility: Flexibility is required in lower bank revetment protection for the same reasons it is required in upper bank pavement protection (see page III-74). However, the need for flexibility is greater in lower bank revetments because of the larger settlements anticipated and because of the more uneven foundation surface below the limits of grading. After a revetment is placed, the channel will often deepen significantly.

In order for the revetment to remain effective, a sufficient length should be originally placed to allow the toe to follow the deepening channel down and continue to provide coverage for the entire slope. This requires both great flexibility and sufficient weight (as discussed below).

Permeability and Filtering Properties: The revetment must have sufficient permeability or drainage features constructed in it to allow groundwater to drain from the bank to the river without creating excessive uplift pressures; yet it must be capable of retaining the foundation materials. These requirements are the same as those for the upper bank pavement discussed on pages II-74 through II-76.

Weight or Anchorage: The revetment must have sufficient inherent weight, sufficient ballast, or be supplied with sufficient anchorage to secure it firmly to the bank and to prevent it from being lifted up by the force of the flowing water. The toe, especially, must have sufficient weight to allow it to follow the deepening channel and to prevent excessive flapping. Securing the revetment by soil anchors may be satisfactory on that part of the lower bank which has been graded, but will not be satisfactory below that point unless no deepening of the channel can be ensured.

Placement Conditions and Methods: The revetment must be capable of being placed in lengths up to 800 feet, in depths of water up to 120 feet, and with currents up to 12 feet per second, on all probable foundation conditions and in normal climatic conditions. The probable climatic conditions are discussed under "Placement Conditions" for upper bank pavement (page II-76). The probable foundation conditions are discussed under the strength requirements above (page II-77).

Lower Bank Indirect Protection

Indirect protection, sometimes called intermittent protection, includes all forms of bank protection which prevent erosion by some means other than direct continuous covering of the surface. Most indirect protection methods fall into two categories: (1) current deflectors, such as vanes and impermeable groins; and (2) current retards, such as permeable dikes, jetty fields, and installations which increase the channel roughness. These forms of protection either deflect the erosive currents away from the structure to be protected, or slow them down sufficiently to eliminate their erosive properties. Their effectiveness is dependent upon the hydraulic conditions of the river, which they, in turn, affect. This interdependence of the fluvial hydraulics and bank protection has been discussed in detail in the previous chapter.

Since hydraulic conditions are somewhat unique at each place where protection is required and since very little is known about the interdependence of hydraulic conditions and bank protection, no particular type of protection is always the most effective. Thus, no criteria were established to meet hydraulic requirements. However, such protection must satisfy certain physical requirements such as durability, strength, flexibility and placement conditions, similar to other forms of submerged lower bank protection. These requirements are discussed in the following paragraphs.

Durability and Strength: The protection must be sufficiently durable and strong (and perhaps heavy) to prevent destruction and displacement by the force of the river current and by the impact of debris carried along by the river. The impact of boats is a serious problem for protection such as permeable retards and surface deflectors which are at the surface. Impermeable deflectors tend to be less subject to such impact because the boats are apt to be carried around them by the deflected current. The materials must retain their durability when submerged and must satisfy the other durability requirements as given previously for lower bank revetment protection (page II-77).

Flexibility: Such protection installations must either be sufficiently flexible to adapt to any foundation settlement or erosion, or sufficiently well keyed into the bank that they will not be undermined by erosion. Because of the nature of such protection, erosion is often quite severe in the immediate vicinity of the installation. For this reason revetment is often placed under and near the structure to prevent or reduce undercutting.

Placement Conditions and Methods: All probable climatic, foundation, current, and channel conditions in the river are discussed under "Placement Conditions and Methods" for lower bank revetment protection (page II-79). Indirect protection structures should be capable of being installed under these conditions.

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PART III

UPPER BANK PROTECTION

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UPPER BANK PROTECTION

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PART III

UPPER BANK PROTECTION

Introduction

The upper bank of the river is defined as that portion which is above water at any particular time. Thus the height of the upper bank will depend upon the river stage, and at "bank full" and higher stages will not exist at all. During those months when riverbank protection is now placed, the upper bank averages about 40 feet in height, and the upper bank protection comprises approximately 25 percent of the total annual area of riverbank protection placed. The investigation of materials and methods that may be applicable for use as upper bank protection is summarized in two chapters. Chapter 1 pertains to artificial riprap protection and Chapter 2 pertains to monolithic protective coverings.

Present Methods

In recent years most upper bank protection has been quarried stone riprap, uncompacted asphalt pavement, or a combination of the two. The physical properties and costs of these two types of protection are discussed below.

Riprap: The present specifications for riprap upper bank protection require a ten-inch layer of stone riprap with a tolerance of plus or minus two inches. The stone shall have a density of not less than 140 pounds per cubic foot. Individual pieces shall weigh no less than six pounds and no more than 125 pounds. No piece shall be larger than 25 inches in any dimension. The approximate gradation is as follows:

75 pounds to 125 pounds	-	10 percent maximum
25 pounds to 74 pounds	-	40 percent to 60 percent
6 pounds to 24 pounds	-	20 percent to 40 percent
Spalls under 6 pounds	-	15 percent maximum

The stone is allowed to have a maximum absorption of two percent (CRD-C-107-60) and a maximum loss of five percent when tested for soundness (CRD-C-137-60). The riprap must be placed so that no more than four square inches of subgrade or filter blanket are exposed in openings between adjacent pieces. A minimum of either three cubic yards or four tons of stone must be placed per 100 square feet of riverbank.

A four-inch thick gravel filter is usually placed under the riprap when the foundation is composed of non-cohesive materials. The only gradation specified for the filter is that it should be graded from coarse to fine.

The present specifications for upper bank riprap protection cover the three criteria of durability, erosion resistance, and filtering properties listed in Part II, Chapter 3. The efficiency of the resulting protection is demonstrated by the fact that very few, if any, bank failures can be directly attributed to riprap failure.

The stone presently used for riprap is procured from quarries in Tennessee and shipped down the river by barge. The shipping costs average two mills per ton mile. The in place cost (excluding the cost of the slope preparation and filter blanket and including transport to Vicksburg) is approximately $4.50 per ton of riprap or $0.18 to $0.20 per square foot of bank protection. The gravel for the filter blanket is obtained at a cost of $2.00 to $4.00 per ton. The cost of grading the slope varies from $0.12 to $0.18 per cubic yard of material excavated.

Uncompacted Asphalt Pavement: When stone suitable for riprap is either unavailable or is too expensive, a hot mix uncompacted asphalt pavement is used for upper bank protection. Sometimes a combination of riprap and uncompacted asphalt is used. The asphalt pavement is placed on the upper portion of the bank and the riprap is placed in the area between the asphalt pavement and the lower bank protection.

The asphalt used in this pavement is a 85 to 100 penetration asphalt cement. The aggregate is a natural sand or a proportioned mixture of sand and gravel, such that not more than 65 percent by weight is retained on the No. 4 sieve, and from one percent to eight percent passes the 100 mesh sieve. Excavated bar run sand is acceptable if it satisfies these specifications.

The asphalt is heated to a maximum temperature of 325 degrees Fahrenheit, and the aggregate is heated to a temperature of 375 degrees Fahrenheit, plus or minus 25 degrees Fahrenheit. They are mixed for a minimum time of 45 seconds and then placed on the slope at a temperature of 225 degrees Fahrenheit to 275 degrees Fahrenheit. The mixture is spread by mechanical means, hand screeding, or raking, to form a smooth layer averaging five inches in thickness. The minimum permitted thickness is 4 1/2 inches. No pavement can be placed during rains or when the surface is covered with ice or snow. Asphalt cement, usually about six percent by weight, is added in sufficient quantity to allow good bonding of the aggregate without any bleeding.

Where required for under drainage, a four-inch thick gravel filter blanket (similar to that used under the riprap) is placed on the bank surface before placing the asphalt pavement.

Although the resulting pavement gives adequate protection against erosion, it is not entirely satisfactory in terms of resistance to mechanical damage, flexibility, and permeability. The pavement apparently tends to harden with exposure resulting in a reduction in flexibility. Its permeability is reduced by the migration of silts and clays into the pores. Thus the pavement often cracks when local adjustments are required and suffers "blow-outs" from high uplift pressures. These failures result in high maintenance cost. A study conducted by the Memphis District indicated that 52 percent of the asphalt pavement had to be replaced by the time it was 12 years old and that maintenance cost for the asphalt protection was three times that for riprap protection (Ref. II-97).

The average in place cost of the asphalt pavement, not including the cost of any required grading or gravel underdrainage blanket, is \$0.22 per square foot. The cost of the asphalt is about \$0.13 per gallon, which, on the basis of six percent asphalt used in an uncompacted five-inch pavement, is equivalent to approximately \$0.03 per square foot. The remaining \$0.19 per square foot cost includes procuring the aggregate, preparing the mix, placing the pavement, and other incidentals. When a four-inch gravel filter is required beneath either the riprap protection or the uncompacted asphalt the additional material cost is from \$0.03 to \$0.068 per square foot.

An interesting comparison between the total costs of riprap protection and asphalt pavement protection was made by the Memphis Engineer District. Their records indicate that the riprap protection has an expected life of 40 years and the asphalt pavement 20 years. They concluded that riprap protection would be economical even if its initial cost was two and one-half times the initial cost of asphalt.

PART III

Chapter 1

ARTIFICIAL RIPRAP UPPER BANK PROTECTION

General

Because of the great transportation distance for quarried stone used as bank protection on the Lower Mississippi River, artificial riprap which could be produced at or near the placement site might cost less. In this study, the category of riprap protection includes any protection formed by unconnected pieces of material. Three general types of artificial riprap have been considered. They are: (1) ceramic riprap, (either manufactured at the placement site or purchased from local manufacturers); (2) soil blocks (stabilized with cement, asphalt, or chemicals); and (3) indurated low-grade rock (upgraded by chemical or other treatment).

Ceramic Riprap

Ceramic materials are used in other countries, especially India and Pakistan, in protecting canal and river banks. The feasibility of using them for slope protection is known to have been investigated twice previously in this country. These applications and studies have indicated that such materials normally have sufficient durability (discussed in Chapter 1 of Part IV) to satisfy the requirements for use in riverbank protection. In this section, previous studies of the possible use of ceramic riprap as slope protection are summarized, and four different products which could be used on the Lower Mississippi River are discussed.

The first two products, vitrified clay masonry units and "Poreen," are or could be available from commercial manufacturers located near the river. The other two products, discussed under "The Pyro-plastic Method" and "Melted Sand," could be produced at or near the bank to be protected.

Previous Studies: The Corps of Engineers is known to have conducted two previous studies on the feasibility of utilizing ceramic riprap as slope protection. The first of these, conducted around 1939, was in regard to slope protection for Kingsley Dam in Nebraska, and was undertaken in cooperation with the Central Nebraska Public Power and Irrigation District (discussed in Ref. III-118). Stone suitable for riprap was not readily available near the dam site, and it was estimated that imported stone would cost approximately $3.50 per ton (personal communication). Ceramic riprap, produced in large sheets and then broken up, was found to have properties in excess of the following:

Crushing Strength	10,000 psi
Absorption (ASTM C-67-31)	7 percent maximum
Soundness (ASTM T-35)	15 cycles (no loss)
Wear (ASTM DA-33)	6 percent
Toughness (ASTM D-3-18)	11
Density	135 pounds per cubic foot
Gross Density of Riprap in place	2300 pounds per cubic yard

The estimated cost of such riprap was $2.50 a ton. The recommendations for using this material, however, were rejected in favor of a slightly less expensive articulated concrete mattress slope protection.

The second investigation (undertaken in 1949 under the direction of the Ohio River Division of the Corps of Engineers) was planned to be more comprehensive. It was to include investigations of the feasibility of using ceramic riprap in the following locations:

a. Portions of the Mississippi River Valley

b. The Lower Mississippi Valley

c. Portions of the Trinity River Basin in Texas

d. The Red River Basin Area (Oklahoma, Texas)

e. The Lake Okeechobee Area in Florida

Unfortunately, this study was never completed. The limited information obtained has been presented in an interim report (Ref. III-118), which includes information concerning the availability of suitable raw materials in the study areas; thoughts concerning the possibility of using ceramic riprap at Garrison Dam (North Dakota) manufactured from local clays and using local lignite coal as fuel; and descriptions and illustrations of some large blocks of fused ceramics produced experimentally for the study. Because this study was primarily directed towards the feasibility of using ceramic riprap as slope protection on dams subjected to large waves, the riprap was produced in much larger sizes than would be required for riverbank protection. Twenty pieces varying in weight from 400 pounds to 1400 pounds, were formed by fusing piles of individual bricks together. The net density varied from 76 pounds per cubic foot to 149 pounds per cubic foot. The gross density was as low as 47 pounds per cubic foot. It is reported that after the writing of the interim report, some of these blocks were moved to a test section along the Ohio River and have suffered no significant deterioration in the intervening 14 years. These test sections are now inundated by the backwaters of the newly constructed Markland Dam.

Vitrified Clay Masonry Units: Vitrified clay masonry units (common clay brick and tile) are manufactured in numerous plants along the Mississippi River. Consideration has been given to the feasibility of procurring low grade brick or tile from these commercial manufacturers for use as riprap bank protection. Because individual units of sufficient weight to be used as riprap are not normally available, several units would have to be bound together. Probably the least expensive method of doing this would be to make small mats which could be handled by light equipment. The mats could be formed by placing copper coated steel wire through holes in the brick or tile. If every other unit were offset so that the wire passes through the opposite end of each successive brick or tile then the wire would only need to be placed in one direction. The bricks or tile would provide tensile strength in the other direction. About ten by ten feet mats would be near the optimum size for handling. The mats could be fabricated on a barge, lifted to the bank with a small crane, and guided into place. Each mat could be anchored in place with reinforcing bars driven into the bank.

A filter blanket would be required to prevent loss of the foundation material. Each mat would have sufficient weight to resist the erosive forces of the river.

It is estimated that the material costs for a brick mat and a tile (12 by 12 by 4 inch units) mat would be \$0.55 and \$0.40 per square foot, respectively. The total in place cost, excluding filter, is estimated to be \$0.65 and \$0.50 per square foot for the brick and tile mat.

Poreen: Poreen is a type of ceramic recently introduced into this country which has a higher density and higher strength than normal vitrified clay masonry units (see page IV-11). Poreen slabs could probably be manufactured in a sufficient size and with sufficient interlocking features to allow them to be placed directly on the slope without connecting them together by wire. A filter blanket might not be required for this application, since a continuous protective covering would be formed to resist erosion through the protection and the connections would provide enough permeability to relieve excess hydrostatic pressure in the bank. Some migration of the foundation material would probably occur. The amount of migration would depend on the tightness of the interlocking. However, it may be cheaper to provide a filter blanket than to develop complicated interlocking features. It is estimated that hand-placed blocks of sufficient weight would result in an in place material cost of \$0.31 per square foot and a total in place cost of \$0.71 per square foot.

An alternate method would be to fabricate mats as described for vitrified clay masonry units on page III-8. It is estimated that the in place material cost would be \$0.35 per square foot and the total in place cost would be about \$0.48 per square foot.

The Pyroplastic Method: Ceramic units manufactured by the pyroplastic method developed by Dr. Mueller (see Chapter 1, Part IV) could probably also be used on an upper bank without connecting individual units together or in mats as discussed on page III-8. These units could be produced at or near the placement site using the silts and clays in the riverbanks. Although a pilot plant would be required to prove the technical and economic feasibility of such a method, Dr. Mueller estimated the material costs, neglecting overhead, would be as follows:

12 by 12 by 6 inch cored mattress blocks (25 lbs.)	- \$2.60 per ton
12 by 12 by 6 inch riprap (70 lbs.)	- \$4.70 per ton
Aggregate	- \$2.60 per ton

He assumed that a rotary kiln producing 360 tons per day would feed four presses (see page IV-16).

It is estimated that the in place cost of mats using the cored blocks would be approximately $0.20 per square foot; and that the in place cost of a layer of dumped riprap at four tons per hundred square feet would be approximately $0.26 per square foot.

Melting Sand: Consideration has been given to the possibility of producing a crude glass riprap by melting sand. It might be manufactured by a process similar to that considered for thermally stabilizing the upper banks (see pages III-34 to III-45). In forming riprap, however, the melted sand would be formed into balls or blocks instead of being placed in a layer on the surface. It is believed that such riprap would be quite brittle and would have numerous shrinkage cracks running through it, giving it a tendency to break apart. However, no known experimentation has been conducted on such a process, and the properties of the resulting product could only be ascertained through testing and pilot operations.

Insufficient information is available to determine the cost of building and operating a plant to produce such riprap. Fuel costs have been estimated to be $1.60 per ton of riprap. This assumes an overall fuel efficiency of 50 percent and a final product weighing about 150 pounds per cubic foot.

Stabilized Soil Blocks

Artificial riprap manufactured from asphalt or concrete is already used occasionally for lower bank protection (see page IV-65). Consideration has been given to the feasibility of using materials to stabilize the natural soils existing at or near the placement site in the form of blocks suitable for use as riprap on the upper banks. Cement, asphalt, and chemicals have been considered as stabilizing agents. More detailed discussions of the use of these agents in stabilizing soils is given in Chapter 2 (pages III-18 to III-33). This section is concerned primarily with the feasibility of producing and using such artificial riprap.

Cement: Soil cement blocks have been used in India for riverbank protection on the Beas River where natural stone was not readily available (Refs. II-95, III-19). These blocks were about two feet square and eight inches thick, and had a density of approximately 125 pounds per cubic foot. The gradation of the soil used in these blocks was as follows:

Clay	8-15 percent
Silt	12-25 percent
Sand	60-80 percent

Only 5 percent cement was added along with sufficient moisture to allow maximum compaction. Five samples were cured for various time intervals and the following strengths measured:

Curing Period Days	Tensile Strength lbs/sq. in.	Compressive Strength lbs/sq. in.
1	9.1	-
3	26.2	217.6
7	38.4	250.4
14	66.2	266.0
28	87.6	280.0

Sacks filled with soil cement have also been used as artificial riprap, both in this country (Ref. III-21) and in West Pakistan (Ref. II-80). In both instances a soil with high clay and low sand content was stabilized with eight percent to ten percent cement.

The normal procedure in making soil cement in the United States is to use soils with approximately the same gradation as was used in making the soil cement blocks but to use more cement -- usually from 10 to 14 percent. The resulting material has high strength, high durability, and high erosion resistance. If properly manufactured and cured, soil cement blocks could probably be used satisfactorily as riprap.

Such riprap could be manufactured either by breaking up hardened soil cement slabs, or casting individual blocks. The latter procedure would probably be preferable. The major difficulty with such a method is that the blocks would have to cure for about a week before they developed sufficient strength to allow them to be finally placed on the slope. The in place material and total costs for dumped riprap, based on four tons per hundred square feet, are estimated to be $0.30 and $0.37 per square foot, respectively. This does not include the filter which would be required.

Asphalt: Asphalt could be another suitable material for stabilizing soil blocks to be used as riprap. Two types of asphalt, cationic asphalt emulsion (see pages III-25 to III-28) and asphalt-sulfur hot mix (see page III-30), might be especially suitable for such construction, because they would allow the blocks to be placed soon after they were fabricated. Asphalt stabilized soil blocks should have some flexibility, especially when new, which might give them a tendency to mold about one another and bind together. Such a protection should have more erosion resistance and inherent filtering properties than just the individual blocks. This tendency could be accentuated by stabilizing the soil with hot mix and placing the blocks on the bank before they have thoroughly hardened. Asphalt stabilized soil blocks could have a density ranging from 110 to 120 pounds per cubic foot, and would probably require from 10 to 15 percent asphalt.

The estimated total in place cost, excluding filter costs, of hand-placed 18 by 12 by 5 inch stabilized blocks with a gross density of 50 pounds per square foot is as follows:

Asphalt - hot mix	- $0.70 per square foot
Cationic asphalt emulsion	- $0.65 per square foot
Asphalt-sulphur hot mix	- $0.64 per square foot

The estimated total in place cost of a 12-inch layer of dumped riprap at four tons per hundred square feet is as follows:

Asphalt - hot mix	- $0.58 per square foot
Cationic asphalt emulsion	- $0.50 per square foot
Asphalt-sulphur hot mix	- $0.78 per square foot

Chemicals: Chemical stabilizing agents could also be used in fabricating stabilized soil blocks for use as riprap. Some of these agents harden very quickly, thus eliminating the storage problem associated with soil cement blocks. However, as mentioned in the following Chapter (page III-33), there is no known chemical stabilizing agent which will provide adequate strength and durability at a cost competitive with asphalt or cement. Therefore, the cost of manufacturing chemically stabilized soil blocks has not been estimated.

Rock Induration

At many points along the Lower Mississippi Valley there are high bluffs composed of weak, low quality rock which, in its natural state, has insufficient strength and durability to be used as riprap. Various methods of indurating this rock, which might sufficiently improve its properties to allow such use, have been investigated. Among these are surface coating, electro-chemical stabilization, and thermal stabilization.

Surface Coatings: The Bureau of Reclamation, at many of its damsites in the Western United States, has the same problem as is experienced on the Lower Mississippi of being unable to readily obtain suitable riprap materials for slope protection. This has led them to investigate several alternative means of providing slope protection, one of which was the possibility of using low quality rocks by coating them with such material as asphalt or indurating their surfaces with chemicals. It is reported that none of these methods were considered successful because, even if the surface of the rock was significantly hardened, the rock still did not have sufficient durability or strength to allow it to meet the requirements of riprap (personal communication). It is concluded that indurating weak rocks by surface application to produce riprap is not feasible.

Electro-Chemical Induration: A great deal of research has been carried on in this country and abroad exploring the possibility of stabilizing soils, especially fine grained silts and clays, by electrical methods (Refs. III-105, 106, 107, 118). Additional research has been directed, most notably by the Russians (Ref. III-116), towards the possibility of indurating weak rock in-situ by electro-chemical methods.

Such electrical or electro-chemical stabilization can occur by one of four processes, as follows:

(1) A simple mechanical stabilization of the soil mass by reducing the natural water content or changing the direction of seepage through application of an electrical potential across it.

(2) A physical stabilization which can occur in certain soils concurrently with the mechanical stabilization whereby the intergranular electrical forces are modified by the application of an electrical potential.

(3) A combination of physical and chemical stabilization whereby the ions moving between the electrodes become bonded to the soil particles, modifying the intergranular electrical forces and perhaps even bonding adjacent particles together.

(4) Primarily a chemical stabilization whereby an electrolytic grouting compound is dispersed through the soil mass by application of an electrical potential.

The first process, mechanical stabilization, provides only temporary stabilizing effects, and its main virtue is that it can be used in clay and silts which cannot be dewatered by other methods. The second process, physical stabilization, occasionally provides some permanent stabilization, depending upon the type of soil, but it is of small magnitude. The third process, combined physical and chemical stabilization, may provide somewhat greater permanent stabilization if it is continued for a sufficiently long time. The fourth process, chemical stabilization, would probably provide the greatest permanent increase in strength and would be most suitable for indurating weak rocks.

The Russians have conducted experiments in applying electrochemical methods to indurate the soil in the walls of oil wells to eliminate the need for casing (Ref. III-116). It was found that sufficient induration was achieved by both combined physical and chemical stabilization (using an electrolyte)and chemical stabilization processes to make such an application technically feasible. In both cases the electrical currents were supplied from electrodes spaced along a pipe placed down in the well. In order to accomplish a more thorough induration, electrodes were alternately moved up and down half the length of one terminal as their polarity was reversed.

In those applications using the combined physical and chemical stabilization process in conjunction with an electrolytic solution, the terminals were moved and their polarity reversed every hour. The total time required for induration was 30 to 50 hours. In those applications using the chemical stabilization process, the terminals were moved and their polarity reversed every 15 minutes. The total time required for induration was four to six hours. Both applications were considered economical in comparison with the steel casing which would otherwise have to be permanently installed in the well. However, it is very doubtful that such a process could be considered economical for indurating soils for use as riprap. In addition, the materials which were indurated were clays (though they were identified as "weak rocks" in the title), and although strength increases from four to five times were achieved by using the processes, the resulting product would probably not be sufficiently durable or strong for use as riprap.

Thermal Induration: The process of thermally stabilizing soils in-situ to increase their bearing capacity has previously been used with fair results by the Russians and Romanians (Ref. III-104, 112, 113, 114). Holes are drilled into the soil mass to be stabilized. Heat is generated at the bottom of a hole by burning a fuel, such as kerosene, which has been fed to the bottom through a pipe as air is forced down the hole. A temperature of from 800 degrees Centigrade to 1000 degrees Centigrade was used resulting in a mass of stabilized soil up to ten feet in diameter.

However, the stabilization primarily results from drying out the soil and involves very little fusing or vitrification. It is mentioned in reference III-113 that if sufficient heat to fuse the soil was generated, a great deal of difficulty was experienced because the melted soil would flow down to the bottom of the hole and extinguish the heat source. Thus it seems impractical to use any such method of thermally indurating weak rocks in-situ to a sufficient degree to make them suitable for riprap.

Induration of weak rocks with existing methods to allow their use as riprap appears to be technically unfeasible. If methods would be improved so that they would be technically feasible, it is doubtful that the process would be economical.

PART III

Chapter 2

MONOLITHIC UPPER BANK PROTECTION

General

As mentioned in the introduction to Part III, the cost of the uncompacted asphalt pavement now used in some areas along the Lower Mississippi River is approximately $0.22 per square foot. Of this total, $0.03 is for the asphalt materials, and $0.19 is for processing and placing. Preliminary investigations indicated that there would be very little hope of, or advantage to, significantly reducing the materials cost in such a pavement. Therefore, all further studies were directed towards finding an inexpensive material which could be processed and placed by a less expensive method.

In this chapter various methods are described by which river sands might be stabilized and placed to provide upper bank pavement protection. "Thermal Stabilization," the feasibility of melting the surface of the bank in order to form a hard erosion resistant skin and "Sheet Protection," the possibility of using a metal or synthetic sheeting or a fabric covering, are discussed.

Cement Stabilization

Portland cement has become a popular agent for stabilizing soils, especially for highway subgrades. It is especially effective in stabilizing coarser grained materials such as sands and coarse silts. It has the advantages of ease of construction, relatively low cost, ready availability of materials, extensive previous usage, and high strength.

Soil cement has also been used as slope protection for earth dams, especially by the Bureau of Reclamation (Refs. III-8, 15, 16, 22, 31), where high quality rock suitable for riprap is not locally available. It has performed very well in such applications because of its strength, durability, erosion resistance, and roughness (which tends to reduce the wave run up).

In addition to normal soil cement which contains from 10 percent to 15 percent cement and just enough water to allow optimum compaction, there is a material called plastic soil cement. Plastic soil cement has a slightly higher percentage of cement and enough water added to make it a fluid of approximately the consistency of thick mortar, much like Gunite. This material has been used in lining irrigation canals. Unlike normal soil cement it can be easily spread on the surface and requires no compaction, making it especially suitable for covering areas where the surface is small, slopes steep, and access to or operation of ordinary compaction equipment is not practical.

Both types of soil cement, as with concrete, do not begin to develop high strengths until they have been left undisturbed a few days. In the construction of the soil cement slope protection at Cheney Dam, the Bureau of Reclamation has specified that after mixing and before final compaction, the material cannot be left unagitated for over one-half hour and the time required for the whole process of transporting, placing, and compacting cannot exceed one and one-half hours. A silty sand with approximately 12 percent cement was used. The in place density averaged about 102 percent of standard Proctor maximum density. Compressive strengths were in the range of 1200 psi for seven-day old samples and 2200 psi for 28-day old samples.

The aggregate for soil cement could be even finer than that presently used for the uncompacted asphalt. The finer bank and point bar sands could be used as aggregate without being dried or heated as is necessary for the hot asphalt pavement.

Either of two construction methods could be used for normal soil cement. The first would be to mix and compact the pavement in place on the bank. The second would be to premix the material on the barge and then place and compact it on the bank. Plastic soil cement could only be premixed and spread or sprayed on the bank. Premixing would probably be preferable for either type of soil cement, because less large equipment would have to be transferred to the bank and a more uniform mixture could be ensured. It is doubtful whether there would be much difference in cost between the two methods.

Soil cement has disadvantages which tend to discourage its use in upper bank protection. The first of these is its rigidity. Unlike the asphalt pavement, soil cement does not have sufficient flexibility to allow it to conform to slight changes in the slope which may occur after it is placed.

A soil cement protection would not have inherent permeability, unless a relatively coarse aggregate were used and the strength of the soil cement were reduced to such an extent that it loses the advantageous properties of durability and erosion resistance.

The Bureau of Reclamation believes that drainage provisions are not needed where the soil cement facing rests upon a relatively impermeable soil, such as clay (personal communication). Their contention is that when the rate of seepage out of the bank is not great, an excessive uplift pressure will not build up behind the soil cement facing. The pressure cannot build up to a magnitude to cause a blow-out, for two reasons. The soil cement facing will be displaced outward a small amount as the pressure increases, relieving the pressure and limiting it to a magnitude below that required for a blow-out. Hairline cracks develop in the facing as it sets (in dam facings these cracks are about eight feet apart), and these cracks allow some seepage through the facing. The Bureau further contends that drainage blankets on fine grained soils are susceptible to clogging at the toe which will aggravate the uplift and blowout problem. A clogged blanket will allow a head of water to build up behind the facing, and a displacement of the facing will not relieve the pressure as is assumed to occur when no blanket is used.

In relatively permeable sand, provisions are needed to allow free drainage of the bank. The higher rate of seepage in such a soil will cause the pressure to increase much faster than displacement of the soil cement facing and seepage through cracks could be expected to relieve the pressure.

In normal soil cement, two general types of drainage provisions are feasible. If the soil cement is pre-mixed the gravel drainage blanket could be used. Weep holes could be installed for either premixed or mixed in place protection. Small pipes could be placed through the soil cement into pockets or trenches of filter gravel. A thin covering of synthetic material would be needed for either type to prevent the soil cement from clogging the filter material, especially if plastic soil cement were used.

Much of the soil on the lower Mississippi River banks appears to fall between the limits of permeability discussed above. The permeability of the soils in the river bank are in many areas more erratic than the permeability of a compacted earth dam. The exact limits of permeability at which drainage is no longer required have not been established. The feasibility of using soil cement without drainage facilities would have to be determined by test sections on banks with soils of various permeabilities.

The soil cement facing at Bonny, Merritt, Poconos, and Cheney dams was mixed in place in six-inch compacted lifts with minimum thicknesses, measured normal to the slope, of from two feet to two feet eight inches (Refs. III-18, 22, 31). The slopes ranged from 2H:1V to 3H:1V. Therefore, the horizontal lifts were six to eight feet wide. Since it is difficult to operate soil cement paving equipment on slopes steeper than 5H:1V, the lift method as used on dams may have to be used on the upper bank of the Mississippi where slopes range from 3H:1V to 5H:1V. An alternative to the lift method would be to operate paving equipment on the slopes with winches anchored above the slopes.

The lift method of placement would require more material than the paving method, to provide a required thickness normal to the slope. Since the width of each lift would have to be at least six feet to allow efficient use of compaction rollers, the minimum thickness normal to a 3H:1V slope would have to be at least 17 inches if placed by the lift method in six-inch compacted layers. If smaller compaction equipment were used so that the minimum thickness measured normal to the slope were 12 inches, then the lift method would require 1.5 times the material required for a 12-inch thick slab on a 3H:1V slope.

It is not known just what thickness of such protection would be required on the upper bank. A thinner protection than that used on dams would probably be more susceptible to cracking. A slab placed on a slope may develop more extensive cracking patterns than the transverse crack patterns observed in protection placed by the lift method. However, if a filter blanket is used on banks with permeable soil, cracking should not reduce the effectiveness of the protection.

At Merritt Dam, the low bid was \$0.62 per square foot in place for a two-foot minimum thickness with 14 percent cement by volume. At Cheney Dam the low bid was \$0.60 per square foot in place for a minimum thickness varying from two feet to two feet eight inches with 12 percent cement by weight. The low unit bid for processing costs at Merritt and Cheney Dam was \$3.50 and \$2.30 per cubic yard, respectively. PCA estimates that processing and placing costs will be bid in the future at \$1.75 to \$2.00 per cubic yard as experience is gained in handling the material.

It is estimated that high quality soil cement slope protection for the upper banks of the Mississippi River would probably cost between \$0.15 to \$0.20 per square foot in place.

Plastic soil cement could probably be applied more cheaply than regular soil cement. Experimental installations by the Bureau of Reclamation of a three-inch thick lining on several canals cost from \$0.08 to \$0.09 per square foot including labor, equipment, and materials, but not including subgrade trimming or contractor's profit. The Bureau estimates that these latter two items would raise the costs to slightly over \$0.12 per square foot. Costs on the Mississippi could be expected to be of this magnitude with a slight increase where drainage facilities are needed.

Asphalt and Coal Tar Stabilization

Asphalt and coal tar are two organic materials having similar characteristics, but differing in their method of production. Asphalt may be a naturally occurring bituminous deposit but is more generally a residue of petroleum distillation. Coal tars are the distillation products of the carbonization of coal. Both can be used in soil stabilization. Coal tar, however, is more expensive, is generally not as available to the Lower Mississippi, and has been the subject of less research as a soil stabilizing agent. Therefore, only asphalt will be considered as a possible agent in stabilizing the upper banks.

Asphalt is available in three basic forms; pure asphalt, cutback asphalt, and asphalt emulsions. Pure asphalt can be heated to above its melting point and mixed with soil or aggregate to produce a "hot mix." This is the form now used in making the uncompacted asphalt upper bank pavement. Cutback asphalt consists of pure asphalt dissolved in volatile solvents. It can be applied to a dry aggregate cold or at slightly elevated temperatures and will harden upon evaporation of the solvent. An asphalt emulsion is an asphalt which has been intimately mixed with water to form, with the help of an emulsifying agent, a water suspension of tiny asphalt globules. These emulsions can be mixed with unheated aggregate and, if normal emulsions are used, will harden upon the evaporation of the water. Research is continuously being directed towards improving all three forms of asphalt. However, for the purposes of upper bank stabilization, the research directed at improving asphalt emulsions is of greatest interest. Pertinent research on the other forms of asphalt will be mentioned in Chapter 3 of Part IV of this report.

Emulsified asphalts have an advantage over hot-mix asphalt, in that they can be used at lower temperatures. They have an advantage over cutback asphalts, in that there is no highly flammable hydrophobic solvent to evaporate. However, normal asphalt emulsions, usually prepared with a crude soap or clay emulsifier, have the disadvantages that they will only harden after the water evaporates (a relatively slow process) and they often do not adhere well to the aggregate.

Research has indicated, that this lack of adhesion is usually due to the fact that each minute asphaltic globule has an inherent anionic or negative charge. Since many aggregates also have anionic surface charges, there is a natural tendency for the anionic asphalt globules to be repelled by the aggregate surfaces.

However, a new form of asphalt emulsion has been recently developed which eliminates these problems associated with the older anionic emulsions. In these new emulsions, the emulsifier has an inherent cationic or positive charge which causes the asphalt globule to be attracted to the anionic aggregate surfaces. These emulsions, called cationic asphalt emulsions, have all the advantages listed above for normal emulsions plus an inherent affinity for most mineral particles which prevents "stripping" and allows the use of damp or wet aggregates.

The process of deposition and solidification of an asphalt emulsion is known as "breaking." One advantage of cationic asphalt emulsions is that their rate of breaking is dependent upon their affinity for the aggregate surface rather than the evaporation of the water carrier. These emulsions can break even when submerged. Although very little is known about the chemistry of producing cationic emulsifiers, emulsions with many special properties can be produced. They can have a very high or relatively low degree of ionization, and they can be produced so that their breaking is primarily dependent upon chemical reactions, time, or their contact with aggregate surface. Cationic emulsions are presently available in three general grades (as listed by The Asphalt Institute): rapid setting, (RS-2K and RS-3K), medium setting (SM-K and CM-K), and slow setting (SS-K and SS-KH). The specifications for these grades are given in Table III-1 (Ref. III-74).

Table III-1

SPECIFICATIONS FOR CATIONIC EMULSIFIED ASPHALTS

Characteristics	AASHO Test Method	ASTM Test Method	Grades: Rapid Setting RS-2K	RS-3K	Medium Setting SM-K	CM-K	Slow Setting SS-K	SS-Kh
TEST ON EMULSION								
Furol Viscosity at 77°F., sec.	T59	D244	-	-	-	-	20-100	20-100
Furol Viscosity at 122°F., sec.	T59	D244	20-100	100-400	50-500	50-500	-	-
Residue from Distillation, % by weight	T59	D244	60+	65+	60+	65+	57+	57+
Settlement, 7 days, % difference	T59	D244	3-	3-	3-	3-	3-	3-
Sieve Test (Retained on No. 20), %	T59[1/]	D244[1/]	0.10-	0.10-	0.10-	0.10-	0.10-	0.10-
Aggregate Coating-Water Resistance Test	-	D244						
Dry Aggregate (Job), % Coated			-	-	80+	80+	-	-
Wet Aggregate (Job), % Coated			-	-	60+	60+	-	-
Cement Mixing Test, %	T59	D244	-	-	-	-	2-	2-
Particle Charge Test[2/]	-	-	Positive	Positive	Positive	Positive	-	-
pH	-	E70	-	-	-	-	6.7-	6.7-
Oil Distillate, % by Volume	T59	D244	5-	5-	20-	12-	-	-
TESTS ON RESIDUE								
Penetration, 77°F., 100 g., 5 sec.	T49	D5	100-250	100-250	100-250	100-250	100-200	40-90
Solubility in Carbon Tetrachloride, %	T44[3/]	D4[3/]	97.0+	97.0+	97.0+	97.0+	97.0+	97.0+
Ductility, 77°F., cm.	T51	D113	40+	40+	40+	40+	40+	40+

1/ Except that distilled water is used instead of sodium oleate solution.

2/ Tested in accordance with paragraph 4.4.4., Particle Charge Test, Interim Federal Specification SS-A-00674C (GSA-FSS) dated August 20, 1962.

3/ Except that carbon tetrachloride is used instead of carbon disulphide as solvent, Method No. 1 in AASHO Method T44 or Procedure No. 1 in ASTM Method D4.

Note: (a) "K" in grade designation signified cationic type.
(b) In Medium Setting Grades -
"SM" indicates sand mixing grade
"CM" indicates coarse aggregate mixing grade.

The rapid setting grades have a high cationic charge and break almost immediately upon contact with any aggregate. They often break so rapidly that thorough mixing is impossible. The medium setting grades do not break as rapidly, but still have sufficient cationic charge to adhere well to the aggregate and to deposit under water. The slow setting grades break quite slowly (often requiring partial evaporation of the water carrier), but still have sufficient cationic charge to adhere well to the aggregate.

The ability of cationic emulsions to adhere to wet aggregate or to break underwater gives it an obvious advantage over hot mixes or cutbacks in possible applications in riverbank stabilization. The sand now used for the upper bank asphalt pavement is usually excavated in a saturated condition. Before the sand can be mixed with the hot asphalt, all the water must be drained off or evaporated, and the sand heated to about 375 degrees Fahrenheit. If cationic emulsions were used, the problem of melting the asphalt, and drying and heating the sand would be eliminated. The sand could be dredged from the river, drained, and mixed directly with the emulsion. By using the same percentage of asphalt as is used in the hot mix, the same properties of permeability and strength should be obtained.

The asphalt emulsion is more expensive than the hot mix asphalt, costing from $0.16 to $0.17 per gallon for an emulsion containing 65 percent asphalt as opposed to approximately $0.13 a gallon for normal hot mix asphalt. The transportation costs would also be higher because of the greater bulk and weight of materials transported for each unit volume of asphalt, but these costs would be partially offset by the elimination of the expense incurred in transporting fuel for heating the hot mix.

It has been computed that if the overall fuel efficiency for heating the hot mix were as low as 25 percent the increased costs of the cationic emulsions and their transportation would be equal to the reduction in the cost of fuel and its transportation.

The use of cationic asphalt emulsions might also allow the protection to be applied more cheaply than the present hot-mix pavement. Two application methods in particular have been considered during these investigations. They are spraying the emulsions directly on the bank and spraying a "solid slurry" mixture of sand and emulsion on the bank.

Asphalt emulsions have been known to penetrate up to ten feet into an open graded sand and a few inches in dense sand (personal communication). It is reported that by using a special cationic asphalt emulsion made from a cutback rather than pure asphalt, the individual globules of asphalt can be made even smaller and are thus able to penetrate sands even more deeply (personal communication). An experimental application of such cationic asphalt cutback emulsions in stabilizing sands was made in the Western United States with the assistance of the Armour Industrial Chemical Company, a large supplier of cationic emulsifiers. They were attempting to develop an inexpensive method of constructing rain catchment basins in isolated desert areas. They found that an impermeable asphalt membrane sprayed on the surface would not stay in place unless the underlying sands were first stabilized to a depth of two or three inches. They were able to effectively accomplish such stabilization by spraying the sand with an asphalt cutback emulsion using "Peneprime" aviation fuel cutback manufactured by the Empire Petroleum Company of Denver, Colorado.

Thus, an asphalt emulsion sprayed directly upon the bank could probably be expected to penetrate and stabilize its surface. The depth of stabilization would depend upon the kind, amount, and concentration of emulsion used. The sand would have to be presaturated before the application because a cationic asphalt emulsion has a tendency to break immediately upon contact with dry sand thus forming an impermeable coating. Theoretically the amount of asphalt could be controlled so that it only served to bind the individual sand grains together and would not greatly reduce the permeability. However, because the surface sand in its natural state is not necessarily any more permeable than that underlying it, it is possible that any reduction in permeability could lead to the creation of uplift pressures and blowouts.

The cost of stabilizing the upper banks by spraying on an asphalt emulsion would probably be comparable to the cost of lining irrigation canals by spraying on a hot asphalt membrane. The Bureau of Reclamation gives these costs as $1.00 to $1.50 per square yard (approximately $0.11 to $0.17 per square foot) in place which includes earth or gravel covering (Ref. III-2). The covering would not be needed for riverbank protection.

The second application method which has been considered is a method used to apply asphalt coatings to automobile and truck bodies to deaden sound. These coatings are applied as a "solid slurry" composed of 80 percent limestone dust and 20 percent asphalt emulsion. The slurry is too thick to spray by conventional methods, but the Gray Company, Inc. of Minneapolis, Minnesota has developed airless positive displacement spraying equipment for this purpose (Ref. III-57). In this equipment the slurry is extruded from the storage cylinder through the hose, and, as it passes through the reduced orifice nozzle, is broken up into a spray. It is possible this or some similar type of unit could be modified in such a way that it could spray a low concentration asphalt emulsion - sand slurry onto the upper banks from a barge. Such an application method would have three advantages:

1. A minimum amount of hand labor would be required.

2. A minimum amount of equipment would have to be transferred from the barge to the bank.

3. A minimum amount of advance preparation of the bank surface would be required.

Plastic netting could be used to reinforce such a coating to eliminate local "blowouts," by placing it on the surface before the slurry is sprayed, or applying slurry, then the netting with another application of slurry over the netting.

The in place material and total costs for spraying on an asphalt sand slurry for a five-inch-thick protection are estimated to be $0.08 and $0.42. This cost does not include any reinforcement. The high in place cost is due to the high cost of the spraying.

Sulphur can be mixed with asphalt to acquire some advantageous properties. Shell Oil Company has found that these properties can be obtained by mixes composed of 10 to 20 percent by weight of asphalt, 5 to 15 percent by weight of filler, 3 to 10 percent by weight of sulphur, and 60 to 80 percent by weight of mineral aggregate. The mix has to be heated to a temperature between 250 degrees Fahrenheit to 325 degrees Fahrenheit. The mixture should not be heated to a temperature greater than 325 degrees Fahrenheit, for at this temperature an undesirable reaction occurs between the asphalt and sulphur. As the mixture cools to between 215 degrees Fahrenheit and 175 degrees Fahrenheit the sulphur crystallizes into very small particles embedded throughout the asphalt matrix. The crystallized sulphur serves as a filler which gives the asphalt increased rigidity and hardness. Marshall stability tests indicate that asphaltic concrete is from four to five times harder at a temperature of 140 degrees Fahrenheit when made with an asphalt-sulphur mixture than when made with pure asphalt.

Thus, the advantages of an asphalt-sulphur mixture over hot-mix asphalt are as follows:

1. Since the mixture begins gaining strength from the crystallized sulphur at a temperature of about 215 degrees Fahrenheit, it has enough strength to be handled at higher temperatures than straight asphalt.

2. The plastic flow properties of the asphalt are greatly reduced, alleviating the tendency of asphalt to deform under load or separate from its reinforcement.

Another asphalt-like material, which might also be classified as a plastic, is "Plasmofalt" (Ref. III-50). Plasmofalt is a resinous bitumen material, mainly composed of a heavy petroleum residue and the aldehydes of sugar. These constituents are combined with a suitable catalyst and heated to form a stiff thermoplastic material very similar to asphalt in behavior and appearance. It can be produced as a hot mix, cutback, or as an emulsion. When used as a hot mix, the aldehydes of sugar can be mixed with petroleum residue in a common asphalt hot mix machine. The cutback is used and prepared similarly to normal asphalt cutbacks. The emulsion is a combination of waste sulfite liquor and Plasmofalt, and it can be made to harden rapidly by the addition of a chromium compound.

Plasmofalt appears to have properties similar to asphalt and could be used for most applications that asphalt is used for, although it may not have the desirable properties which are peculiar to cationic asphalt emulsion. Plasmofalt may be superior to normal asphalt in some respects such as creep and tensile strength, but the material has not been used or tested sufficiently to ascertain its potential. What testing it has received does not indicate that it is of poorer quality than asphalt. It should be competitive with asphalt in cost. A small continuous Plasmofalt emulsion plant is reported in production in Albuquerque, New Mexico and a franchise has been granted a mid-west firm. Otherwise, only batches of less than 1000 gallons have been produced.

Chemical Stabilization

A great deal of research has been conducted during the past 20 or 30 years into the possibility of stabilizing soils with chemicals. Some of the most significant and productive of this has been done by the Massachusetts Institute of Technology under contract to the Waterways Experiment Station, U. S. Army, Corps of Engineers (Ref. III-90 to III-94). The Civil Aeronautics Administration tested over 100 different compounds (Ref. III-89). However, none of this work has produced any chemical soil stabilizer which would be significantly superior to or more economical than asphalt or cement in stabilizing sands.

Since there is not a chemical stabilizer at present that is satisfactory for use in stabilizing the upper banks, the many chemicals whose effectiveness as soil stabilizing agents have been investigated will not be summarized. Other sources than those normally associated with soil stabilization (such as industrial research departments) were contacted to determine if they might possibly have succeeded in discovering any new stabilization methods which might be used in the upper bank protection. In general, the conclusions of these investigations were negative. Certain chemicals, such as various phosphorus compounds, have demonstrated an ability to stabilize fat clays more economically and effectively than cement or asphalt because of their ability to interact with the clay minerals. However, there seems to be no advantage in using chemicals for the stabilization of surface sands. Such stabilized sands will generally have less strength than soil cement or less flexibility than the asphalt stabilized sands, and will be significantly more expensive.

Attempts were made to determine whether materials of a lower quality than normally considered adequate for soil stabilization could be used to stabilize the riverbanks. In this respect, discussions were held with the Quaker Oats Company, a major producer of furfural resins, to investigate the possibility of using decomposed agricultural products in which the furfural resins had been activated but from which they had not yet been distilled. It was found that, at this stage of processing, the resin occurs in small quantities (approximately five percent). If it were possible to make all of it react, the resin would hardly be sufficient to adequately stabilize even the decomposed waste, much less any soil that was added.

If a chemical were found to be effective in stabilizing the riverbank it could probably be economically applied by either of the two methods, direct spraying or "solid slurry" spraying, discussed above for cationic asphalt emulsions. However, since chemicals are generally more expensive than other stabilizing agents and offer no technical advantage, it is concluded that chemical stabilization is not feasible for riverbank protection at this time. With the rapid advancement of technology it is highly possible that an epoxy may be developed, which if produced in sufficient quantity would be economical and of such a viscosity that it could penetrate the river banks causing little or no reduction in bank permeability and would add sufficient strength to resist erosive forces.

Thermal Stabilization

Soils can be thermally stabilized to increase their strength, bearing capacity, durability, or erosion resistance. The simplest form of thermal stabilization, that of sun drying, has been used for many centuries in making adobe. However, the hardening which occurs during this process is not permanent and the materials will usually revert to their former condition when exposed to a sufficient amount of water for long periods of time.

To be feasible for application in stabilizing riverbanks, thermal stabilization must go beyond the simple drying of the materials and involve some permanent fusing or vitrification. The feasibility of using prefabricated vitrified materials (or ceramics) has been discussed in the previous chapter. The possibility of thermally stabilizing the bank materials in situ is discussed below.

Previous investigations concerning the possibility of thermally stabilizing surface layers of soils in situ have been conducted by:

1. The U. S. Army and U. S. Navy in regard to stabilizing beach sands by the thermite process (Ref. III-108).

2. The Australians in regard to stabilizing silts and clays for concrete aggregate or pavement subgrade (Refs. III-109, III-110).

3. The U. S. Army in regard to producing riprap for slope protection (Ref. III-117).

Because some of these investigators concluded that such a process would be economically and technically feasible, it was decided that it should also be investigated for use on at least the upper banks of the Lower Mississippi River.

The preliminary approach to this problem was to compute the amount of heat required and necessary fuel costs to melt the bank sands. The values used in the computations are given in Table III-2.

Table III-2

SOIL VALUES USED FOR THERMAL STABILIZATION

Symbol	Item	Value	Reference
γ_s	Saturated density of sand	1.13 lbs/cu. ft.	B-9
w^1	Saturated water content of sand	45%	B-9
γ_d	Dry density of sand	78 lbs/cu. ft.	Computed
S_r	Natural degree of saturation	70%	Assumed
T_1	Natural temperature	50°F	Assumed
T_2	Melting temperature of sand	2550°F	B-4
K	Thermal conductivity of sand	0.2	B-1
C_s	Specific heat of sand	0.2 BTU/lb.	B-3
C_w	Specific heat of water	1.0 BTU/lb.	B-1
H_v	Heat of vaporization of water	970 BTU/lb.	B-4
γ_f	Density of fused sand	160 lbs/cu. ft.	B-3

The heat required to melt one cubic foot of dry sand is:

$$(T_2 - T_1)\ (C_s)\ (\gamma_d) = 39{,}000 \text{ BTU}$$

The heat required to evaporate one pound of water is:

$$(212 - T_1)\ (C_w) + H_v = 1132 \text{ BTU/lb.}$$

Thus the heat required to evaporate the water from a cubic foot of soil is:

$$(1132)\ (S_1)\ (W^1)\ (\gamma_d) = 27,800 \text{ BTU}$$

Total heat required to fuse a cubic foot of soil would be:

$$39,000 + 27,800 = 66,800 \text{ BTU's}$$

Thus, if six inches of natural soil are to be fused, approximately 33,000 BTU's of heat per square foot of riverbank would be required (assuming 100 percent efficiency). This estimate is probably low because neither the increase in the specific heat at higher temperatures nor the heat of fusion of the soil has been taken into account. Lembe (Ref. III-5) has estimated that approximately 3,000,000 BTU's of heat per cubic yard (about 55,000 BTU's per half cubic foot) would be required to vitrify clays.

The amount of heat that would have to be applied to the soil to obtain the quality of protection desired and the quality of the protection itself will depend upon the methods used to apply the heat. Heat can be applied in at least three ways: (1) apply heat to the bank surface, (2) mix a combustible material with the soil that will provide heat when ignited, (3) apply heat in a portable kiln.

Surface Heat Application: If heat is applied to the surface of the soil, significant heat losses will occur by heat transfer to the subsoil and by heat losses to the air. The heat losses to the air will depend to a great degree upon the manner in which the heat is applied. If an open flame is used such losses will be quite high, whereas if expensive equipment is used to confine the heat (as was done in the Australian applications), the losses will be lower. There will probably be an optimum point where any increase in equipment cost will no longer result in an equivalent saving in cost of heating.

An estimate can be made of the amount of heat which would be lost to the subsoil. The Permafrost division of the Corps of Engineers has conducted a great deal of significant research into thermal effects in soils during the past 10 or 15 years. Some of the computational methods used in this work (Ref. III-115), based on the principles of conduction through solids, can be used to qualitatively estimate what some of these losses might be.

The temperature distribution in a solid at any time, t, resulting from an increase in temperature at the surface of the solid can be approximated by a parabola as indicated in Figure III-1. The temperature, T, at any time, t, and at any depth, Z, below the surface of the solid can be given by the equation:

$$T = \Delta T \frac{(Z_1 - Z)^2}{(Z_1)^2}$$

The rate at which heat enters the solid will be:

$$q = K \frac{2 \Delta T}{Z_1}$$

The total quantity of heat which has entered the solid at time t is the area under the parabolic curve in Figure III-1, multiplied by the specific heat (C) and the density of the solid, (γ_m). Thus:

$$Q = 1/3 \quad (\Delta T) \quad (Z_1) \quad (\gamma_m) \quad (C)$$

These equations are applicable to heat transfer in porous media as well as solids if the proper values of K and C are chosen. Those values will vary, for any particular soil, with density and water content.

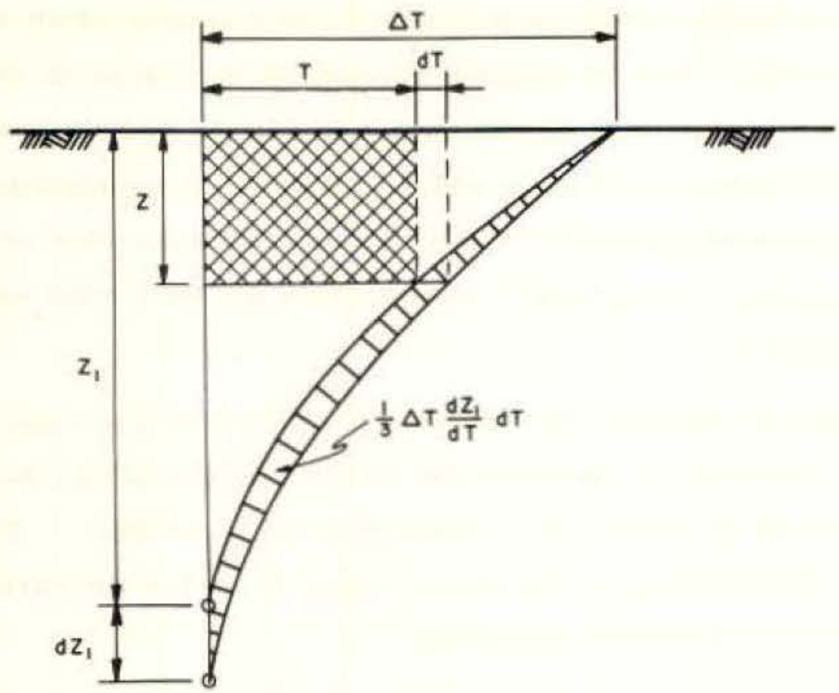

Figure III-1. TEMPERATURE DISTRIBUTION IN SOIL

If Z is the depth to which it is desired to stabilize the soil, and T is the temperature required for fusion, a qualitative estimate of the efficiency of the thermal stabilization process can be obtained by dividing the amount of heat theoretically required to fuse the soil by the quantity of heat that has entered the soil at the time fusion is theoretically complete.

$$\text{Eff} = \frac{(T) \quad (Z) \quad (\gamma\, m) \quad (C)}{1/3 \quad (\Delta T) \quad (Z_1) \quad (\gamma_m) \quad (C)} = 3 \quad \frac{(T) \quad (Z)}{(\Delta T) \quad (Z_1)}$$

Maximum efficiency is obtained by minimizing the product of ΔT and Z_1. Since these are interdependent variables, and Z_1 decreases with increasing ΔT, there will be an optumum ΔT which will result in the highest efficiency. This optumum value will depend upon the thermal diffusivity (α) of the soil, which is the rate at which the soil undergoes temperature change upon heating or cooling. A curve showing the probable relationship between efficiency of the soil and ΔT for the sands in the upper banks is given in Figure III-2.

A problem that was ignored in the theoretical discussions of the heating efficiency of soil was the difficulty of fusing the soil to more than an inch or two of depth. As the surface grains are melted, a thin sheet of liquefied soil is formed. The melted sheet prevents hot gasses from entering the soil by convection. Thus, the melted soil must be the direct source of heat for the underlying soil. This means that it must be kept in the molten state at temperatures higher than the melting temperature of the soil for a considerable amount of time to allow the soil grains to be melted at the depth that would be required for bank protection. The fuel costs will increase and the placement rate will decrease because of this problem.

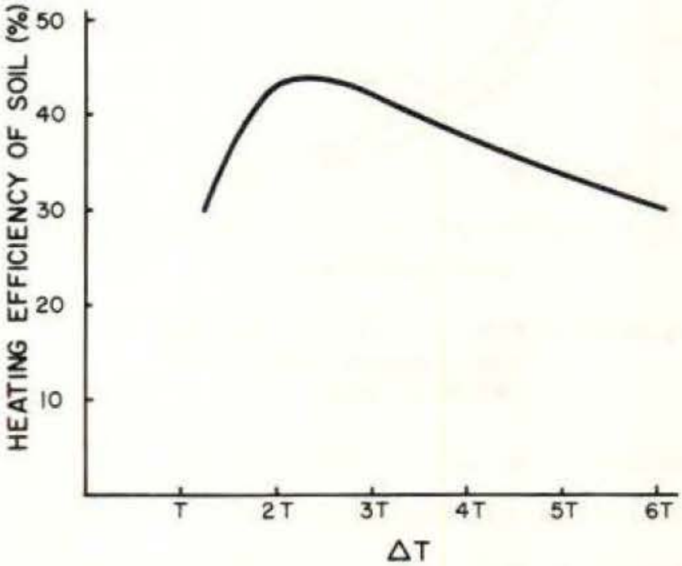

Figure III-2. HEATING EFFICIENCY OF SOIL BY SURFACE HEAT APPLICATION.

A machine of the type used by the Australians could be developed to heat the soil by applying a flame and hot gases to the surface of the upper bank. Commercial fuels, such as natural gas and fuel oil, could be used as a source of heat. Assuming the heating value of 144,000 BTU's per gallon and a cost of $0.07 per gallon for fuel oil, and a heating value of 1050 BTU's per cubic foot and a cost of $0.40 per thousand cubic feet for natural gas, the variation of fuel costs versus heating efficiency of the soil for stabilizing a six-inch layer of natural soil per square foot of riverbank is given in Figure III-3. The curves assume a 100 percent fuel efficiency and a heat requirement of 32,000 BTU's per square foot for fusing the soil.

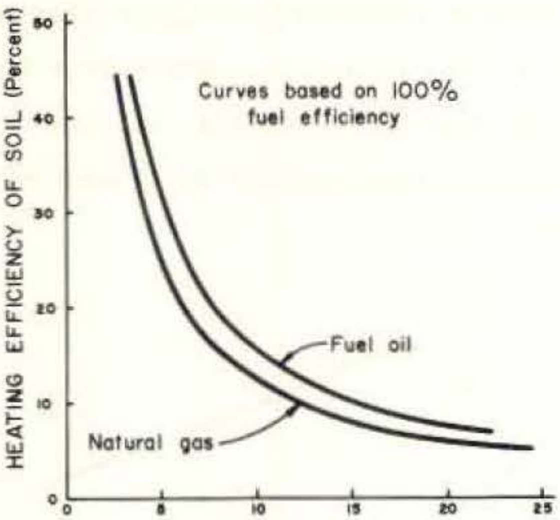

Figure III-3. FUEL COST FOR FUSING ONE SQUARE FOOT, SIX INCHES THICK.

The sun provides a free source of heat energy that can be collected and focused by a system of mirrors or lenses. The amount of radiation from the sun per square foot that can be expected at various times of the year at the latitude of Vicksburg (Ref. B-1, p. 12-115) is given in Figure

III-4. It can be seen that the maximum rate is about 3000 BTU's per square foot per day on a horizontal surface. Although a mirror which could be adjusted to follow the sun during the day would probably collect more energy, such a mirror would still have to have an area of ten square feet or more to collect enough energy on a clear day to stabilize one square foot of river surface in eight hours with an efficiency of 100 percent. Such an application is thus beyond reasonable consideration.

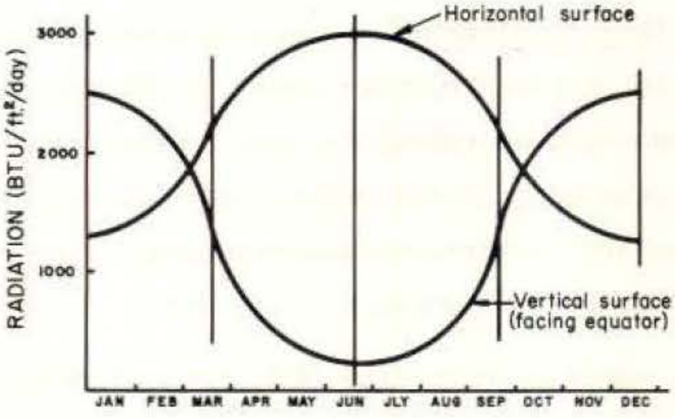

Figure III-4. SOLAR RADIATION AT VICKSBURG.

Laser rays (light amplification by stimulated emission of radiation), a very new development in physics, are another energy source which might be used to apply heat to the bank surface. A form of energy (usually light or electrical) is converted into a very highly concentrated beam of light. These high energy light beams convert into heat energy when they meet any absorptive opaque surface. They can be of such high energy and concentration that they can instantaneously melt holes through diamonds. The science of lasers being very young, there is a continuous influx of new developments. The use of lasers in riverbank stabilization is obviously not feasible now because of the high cost of the individual units and the low efficiency of those emitting the required amount of energy. Lasers have been developed which, by amplification, are able to produce over 500 megawatts of energy. These units have an efficiency of only one or two percent (Ref. III-111). Lower energy lasers (less than a watt) have been produced with efficiencies as high as 50 percent. Further developments in this field can be expected to increase both the efficiency and energy levels of individual units. It is believed that their use in riverbank stabilization will not be feasible for a good many years, if at all.

Internal Heat Application: In many sintering processes, a combustible material is mixed with the soil to enable the heat to be distributed more uniformly throughout the soil. External heat is applied in a sufficient amount to ignite the combustible material and sustain combustion until the soil achieves the desired state. The combustible materials require oxygen to burn or give off heat.

If such a material were mixed with the soil on the river bank, it could be ignited by applying heat to the bank surface. The amount of heat required to sustain combustion would depend on the percent of combustible material in the mixture and the properties of the combustible material used.

A combustible material that could possibly be used is jellied fuel, such as napalm. It requires access to oxygen if its combustion is to be self-sustaining. Thus, it is not known to what depth the napalm would burn in a napalm-soil mixture without requiring oxygen and a surface application of heat to sustain combustion.

A combustible material that does not necessarily require free oxygen from the air is powdered aluminum. It can obtain bound oxygen from oxides and other materials in which oxygen is loosely bound to sustain its combustion (thermite method). There is an exothermic reaction in which a considerable amount of heat is generated. Clay contains oxygen between the layers of its sheet-like structure that is more loosely bound than the oxygen in the basic silicon dioxide tetrahedron. Thus, a clay will provide some oxygen to help the thermite sustain its reaction, whereas sand will require the mixture to be heated to the high temperature at which the oxygen is released from the relatively stable silica tetrahedron.

Electricity could be used to ignite a mixture of soil and a thermite type material, or it could possible be used to fuse the soil without adding any combustible material. It is being used to fuse soil in some ceramic processes. The fusing of the riverbank soil would be accomplished by having an arc jump between two electrodes inserted into the soil. It is questionable whether or not such a process would work unless the soil is nearly dry. The almost instantaneous evaporation of water in the soil would probably produce such a force that it would blow the soil off the surface. However, lower voltages could be used if the electric arc were used to ignite a soil-thermite mixture. The amount of heat required to ignite the mixture may be low enough so that the soil would not be disrupted.

Internal heat application to the soil should be a more efficient process than surface heat application because less heat will be lost and the soil will be fused much faster. It should be possible to fuse the soil to a required depth.

<u>Confined Heat Application</u>: Many of the problems associated with surface and internal heat application can be resolved by applying heat to the soil in a portable confined furnace, such as the air suspension kiln (fluidized bed calcining unit) shown on Figure IV-3 (page IV-15). The

soil could be removed from the bank, heated in the kiln, and replaced on the bank in a layer.

This method would utilize the heat more efficiently than surface or internal heat application methods. Better control could be maintained over the melting process and a more uniform fused material could be obtained. The thickness of protection required could be obtained without a large decrease in efficiency as with surface heat application.

Quality of Protection: The strength, durability, and permeability of the fused soil will depend on the temperature attained in the fusing process. If enough heat is applied to just melt the surface of the soil grains, then the soil grains will be bonded together to form a fused soil with some inherent permeability. If the temperature is carried past the point of incipient fusion, the body of the soil grain will melt. The soil will reach a molten state and then be transformed to a dense, solid state as it cools. Such material is impermeable and is actually a form of glass.

In the Australian experiments referred to previously, the material which was being vitrified was a silty clay, probably similar to the silt overburden. The following description of the processes which occurred in these applications when excessive heat was applied and the vitrification process continued beyond that required for their use, was given by Irvine in Reference III-109:

> "If the heating is carried to excess, the soil assumes the consistence of treacle and will adhere firmly to an iron bar poked into the furnace. At this stage the material tends to become vesicular and to lose its density. If it is then subjected to a continued excess of heat it is liquefied and flows over the surface like lava, forming a sheet which closes the voids and prevents further penetration of heat by convection.

If the heating is still further continued, the liquefied material gradually loses its vesicular character and will solidify on cooling into a dense black and very hard product."

Although the strength and durability of the material will depend on the degree to which the soil is vitrified, a material could probably be formed that would have adequate strength and durability for use in river-bank protection. Although the material itself should have these properties, a monolithic protection composed of fused soil may not be durable and strong. A monolithic protection would probably be cracked due to thermal shock and differential cooling. Thus, the protection would not be rigid, but would be composed of large slabs of sufficient weight to resist the erosive forces of the river. The degree of cracking will depend on the method of applying heat and the control exercised over the process. Cracking could be controlled better by heating the soil in a portable kiln than by an internal or surface in-situ application of heat.

If extensive cracking is allowed, the protection would not retain cohesionless foundation soil. Although the fused soil could have some inherent permeability, it might not be adequate to allow the bank to drain freely where the bank soil is permeable. A filter blanket could not be placed beneath the protection.

Thus, it appears that drainage provisions (similar to those discussed for the use of soil cement on page III-21)installed in a relatively uncracked slab would be the only means of providing adequate upper bank protection by thermal stabilization. However, the durability, protective ability and economy of such protection will depend greatly upon the method used for stabilization, and could only be assessed by observing experimental installations and testing.

Sheet or Fabric Protection

Another method of continuous protection for the upper bank surface which has been given consideration is the use of sheeting or fabrics. These could either be synthetic materials as discussed in Chapter 2 of Part IV, or metallic materials as discussed in Chapter 3 of Part IV. Many of the synthetic materials have a very low cost (see Table IV-2), varying from two to ten cents per square foot, and the installation costs would also probably be low. The metallic materials have a high cost. No savings could be expected through their utilization.

Synthetic Films and Fibers: The synthetic materials have some properties which discourage their use in such an application. The first of these is the tendency exhibited by most of them to suffer severe deterioration when exposed to sunlight. Materials having this tendency would be unsuitable, unless adequately protected. Some synthetic materials, especially the elastomers, do not suffer as severely from such deterioration. However, these materials still have the low strengths, tearing resistance, and puncture resistance typical of all synthetics when compared to the materials normally used in Civil Engineering construction. The low melting point of some of the synthetic materials and the properties mentioned above would probably cause a vandalism problem, since the protection would be exposed most of the time. The materials would be susceptible to mechanical damage by boats or livestock. It is an economical question whether the low in place cost of these materials would be offset by high maintenance costs.

If synthetic films of fibers were to be used, they could be easily secured by one of three methods. They could be weighted down by some superimposed weight, such as concrete blocks, they could be fabricated into mattresses which would be filled with a sand ballast, or they could be anchored to the slope with soil anchors. The latter two methods are discussed in more detail in Chapter 2 of Part IV.

The fabrics could have inherent permeability and would not require a drainage blanket. The weave of the fabric can be made dense enough to prevent migration of the foundation material. The films, on the other hand, have insufficient inherent permeability and would require drainage provisions. A gravel drainage blanket could be used beneath the film or perforations could be made in the film. Some synthetic materials are better suited for perforated film than others. A measure of their suitability is their tearing strength as listed in Table IV-2.

It is estimated that placement costs could be as low as \$0.03 per square foot for a single sheet protection placed by a method similar to Roll-Placement Method II, (see Chapter 2, Part IV). If the fabric or film could be secured to the slope with soil anchors, the estimated in place cost is from \$0.15 to \$0.25 per square foot.

Asphalt Mattress: Pre-fabricated asphalt mattresses are available commercially. These mattresses generally contain fibrous materials which make them more durable and flexible than the uncompacted asphalt pavement presently used in upper bank protection. One such product, Hydromat (made by W. R. Meadows Inc.), is manufactured in thicknesses up to one-half inch. It is described as a core of blended air-blown asphalt, organic fibers and mineral fillers, a covering of asphalt-saturated felts and an exterior coating of weather resistant oxidized asphalt (Ref. III-58). The material is placed in four foot wide strips that can be joined by laps made in the direction of river flow and sealed with cold asphalt mastic or hot asphalt cement. The one-half inch material costs about $0.12 per square foot.

Since these mattresses have insufficient inherent permeability they would have to be punctured or underlain with a drainage blanket to allow the foundation to drain. The mattress would have to be anchored to the slope or weighted down with ballast, for it would be too light to be stable in place.

Metal Sheeting: Light gauge aluminum alloy or galvanized steel sheeting could be placed on the upper banks. Metal sheeting would have to be used with a drainage blanket to help overcome some of its obvious disadvantages. It is not flexible enough to conform to any but minor changes in the foundation. Thus, the sheets would have to be joined in a manner that would prevent loss of material from the foundation. In order to provide an economic protection, the sheeting would have to be thin and could not be used with a double thickness. The protection would by very lightweight and could be placed from large rolls. Because of its relatively high cost it could not be used in a mattress form with sand ballast.

It is estimated that metal sheeting would have a total in place cost of from $0.40 to $0.70 per square foot if placed by a method similar to Roll Placement Method II (see Chapter 2, Part IV). A possible alternative is to weave a flexible mattress from wire as discussed in Chapter 3, Part IV. The placement cost for a wire mattress is estimated to be as low as $0.03 per square foot, assuming a method similar to Roll Placement Method II is used. Soil anchors could be used to secure it to the bank.

PART III

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PART IV

LOWER BANK PROTECTION

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LOWER BANK PROTECTION

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PART IV

LOWER BANK PROTECTION

Introduction

The lower bank of a river is defined as that portion which extends from the water surface down to the thalweg at any particular time. Although the amount of lower bank is thus dependent upon the river stage, there is a portion (that below minimum low water) which is always classified as lower bank. During the season when lower bank protection is presently placed the river is near its low stage and the top of the lower bank is an average of 40 feet below the top of the natural riverbank. The depth of the lower bank varies from 40 to 120 feet, and revetment lengths up to 700 feet are required to provide complete protection. The average length of the lower bank protection is 350 feet. Since the thalweg at concave bends tends to deepen after the bank has been protected, the lower bank revetment may be extended some distance beyond the existing thalweg in order to allow it to provide protection for the entire bank after this deepening has taken place. Lower bank protection comprises about 75 percent of the total bank area protected and about 90 percent of the total initial cost.

Presently, the articulated concrete mattress is the only revetment protection used on the lower banks of the Lower Mississippi River. This mattress, its construction, physical properties, and cost are discussed in this introduction. The first three chapters in this part of the report discuss various other materials which might be used in revetment type protection of the lower bank. In Chapter 1, the technical and economical feasibility of using ceramics strung into mattresses is discussed. In Chapter 2 the various properties of synthetic sheeting and fabrics which might affect their use in lower bank revetments are discussed, along with various possible methods whereby such revetments might be fabricated and placed. In Chapter 3 various other materials which might be used in revetment type protection are discussed. Materials which could be used in constructing various indirect (or intermittent) protection are discussed in Chapter 4. Appendix IV-A includes a discussion of research into, and testing of, the suitability of the Mississippi Valley silt and clay deposits for ceramic processing.

Articulated Concrete Mattresses: The articulated concrete mattresses are fabricated on the launching barges from precast units which are 25 feet long and 4 feet wide. These units are called "squares" because each covers an area of 100 square feet. Each square is composed of 20 concrete blocks, approximately three inches thick, four feet wide, and 14 inches long, connected together by three corrosion resistant copper coated steel wires having a breaking strength of 4000 pounds each. The individual blocks are separated by one inch interstices and have two strands of reinforcing wire running through their width. The reinforcing wire is continuous in each direction and forms shallow loops at the ends of the squares and at the edge of each block in order to facilitate the fabrication of the units into the final mattress.

The squares are cast by private contractors in seven casting fields owned by the Corps of Engineers located along the river from Caruthersville, Missouri to St. Francisville, Louisiana at intervals ranging from 75 to 126 miles. The fields are adjacent to the river and allow easy loading of the precast squares on barges. Some of the casting fields are potentially susceptible to flooding during high water, but this is apparently a rare occurrence and does not limit the production of mattresses.

The precast units are transported to the concave bank requiring protection where they are moved to the deck of the launching barge, connected together with twist wires, and fastened to launching cables by wire rope clips to form the final mattress which is approximately 90 feet wide. The launching cables are secured to the launching barge and anchored in the bank to form a cradle for support of the mattress when it is placed through the water. The fabricating and launching of the mattresses is a continuous operation. After each row of precast units is fastened to the preceding row and to the launching cables, the launching barge moves away from the bank a sufficient distance to allow most of the row to slide off the launching deck into the water so that room is created for fabricating the next extension. This process is repeated until the required length of mattress has been fabricated. That part still hanging in the water is then lowered to the bottom, the launching cables are cut at the barge end and the launching barge returns to the bank to begin the fabrication of the next mattress. The placing proceeds upstream with each mattress overlapping the upstream edge of the one placed immediately before by a minimum distance of five feet. The mattresses are fabricated and launched by the Corps of Engineers.

The articulated concrete mattresses lower bank revetment has been under development for use on the Mississippi River since 1915. Thus, the present mattress design probably provides the optimum combination of low cost and efficacious physical properties. Several modifications of this mattress have been experimented with and other types of mattresses have been tried, but it has always been concluded that the present articulated concrete mattress offers a better combination of rapid and easy placement, good protection, and low cost.

In regard to the criteria listed in Chapter 3 of Part II, the present mattress is durable, flexible, permeable, and apparently has sufficient strength and weight. One disadvantage is that it does not provide adequate filtering of the foundation, occasionally resulting in a tendency for the foundation materials to be sucked out through the interstices and the mattress to be undermined. Efforts at reducing these interstices from eight percent of the mattress area to three percent of it (the V-type mattress) apparently proved unsuccessful because of the difficulty experienced in launching the mattresses.

The durability of the concrete and the reinforcing fabric is good. However, mostly because of migration of the river channel and bank failures, maintenance is required. Lower bank maintenance now involves 225,000 out of the 600,000 squares of articulated concrete mattress placed each year (personal communication). Haas states that "The overall average cost of annual maintenance is $15,000 per mile of operative revetment" and gives the total annual maintenance and reinforcement of operative revetments factor as three percent (Ref. II-97).

The tensile strength of the mattress is provided by the reinforcing wires described above. Assuming that the connections between squares are as strong or stronger than the reinforcing itself, the tensile strength along the length of the mattress is (3)x(4000) = 12,000 pounds per concrete block, which is equivalent to 3000 pounds per foot or 250 pounds per inch. The permeability of the mattress is provided by the openings between adjacent concrete blocks which amount to about eight percent of the gross mattress area. The submerged weight of the mattress is about 20 pounds per square foot.

The cost of the mattress is approximately $0.40 cents per square foot in place. This cost can be divided into two parts, materials and placement. The materials cost is the cost of the precast articulated squares to the Corps of Engineers at the casting field. This has been reported to be about \$0.186 per square foot as indicated in Table IV-1. The remainder of the \$0.214 per square foot includes transportation, placement, etc.

Table IV-1

COST OF MATERIALS
IN THE ARTICULATED CONCRETE MATTRESS
(Per 100 Square Feet)

Item		Cost
1. Cement		\$ 3.50
2. Reinforcing Wire		4.80
3. Labor and Other Materials		5.82
4. Clips, Launching Cables, etc.		2.96
5. Casting Field & Form Rental		0.15
	Subtotal	\$17.23
6. 8% for Engineering, Supervision, and Administration		1.37
	Total	\$18.60

PART IV

Chapter 1

CERAMIC MATTRESS REVETMENTS

General

Ceramics are materials which have been fused together (or vitrified) by the application of heat. Extensive research has been conducted during this study into the possibility of utilizing such materials in riverbank protection. A previous study on this subject is discussed in Chapter 1 of Part III along with the possibility of using ceramic riprap for upper bank protection. In this chapter, the possibility of stringing ceramic blocks on wires to form mattresses for lower bank protection will be discussed, along with the properties, costs, and sources of some ceramic materials which might be so used.

Ceramics produced for Civil Engineering applications are typically quite strong and durable, and are usually produced by vitrifying suitable clays and silts. Ceramics can have various properties depending upon the manufacturing methods employed, the quality of the raw materials, the degree of vitrification, and the finishing process.

Ceramics are available from commercial manufacturers in standard forms which could be used in riverbank stabilization. Two types of commercially available materials have been considered in this study. The first is the normal "structural clay product" which could be procured from a number of existing manufacturing plants located in the Lower Mississippi River Valley. The second is a special high density, high strength ceramic recently introduced into this country. These two types are discussed in the section entitled "Commercially Available Ceramics."

The possibility of producing ceramics from the silts and clays available along the river at or near the bank to be protected has also been investigated. The suitability of the readily available fine grained deposits for ceramic processing was determined, and possible processing methods were investigated. Appendix IV-A includes a summary of previous investigations of these materials and a report of the testing conducted by Dr. James I. Mueller for this study.

Commercially Available Ceramics

Many different types of ceramics are commercially available to serve many different purposes. However, this study has concentrated on two types which are produced primarily for civil engineering construction and which would probably be the least expensive while still providing adequate physical properties. The two types studied are common vitrified clay masonry units and a new material called "Poreen."

Vitrified Clay Masonry Units: Vitrified clay masonry units are produced throughout the United States, and there are many plants located in or near the Lower Mississippi Valley. The properties of these units vary considerably as do the available shapes and sizes, depending on the particular manufacturer and the raw materials he uses. However, through the efforts of the National Bureau of Standards, the A. S. T. M. and several professional and trade organizations, many of these variations have been reduced. Standard terminology, testing procedures and specifications have been instituted (Ref. IV-1). These standardizations greatly facilitate the use of such materials in civil engineering construction. Special ceramics, with special properties, can be produced also.

However, only the properties of standard units have been investigated because of their general availability and low cost.

Vitrified clay masonry units are divided into two classifications; brick and tile. A brick is defined as a masonry unit "whose net cross-sectional area in every plane parallel to the bearing surface is 75 percent or more of its gross cross-sectional area measured in the same plane." A tile is defined as a masonry unit "whose net cross-sectional area in any plane parallel to the bearing surface is less than 75 percent of its gross cross-sectional area measured in the same plane."

Bricks are graded according to their resistance to freezing and thawing and their proposed use. The three grades are, in order of decreasing weathering resistance, SW, MW, and NW. A MW grade, or perhaps a SW grade, would be required for riverbank stabilization. There are also several types of structural tile, three of which are of primary interest to this study. These are load-bearing wall tile, non-load bearing wall tile, and floor tile. Load-bearing wall tile is assigned to one of two grades, again depending upon their weathering resistance and proposed use. These are grade LBY and grade LB. Non-bearing wall tile is available in only one grade which is of somewhat poorer quality than either grade of load bearing wall tile. Floor tile is available in two grades FT1 and FT2.

The physical properties of all vitrified clay masonry units which are of greatest interest in relation to the feasibility of their application in riverbank stabilization are: (1) Density; (2) Strength; (3) Water Absorption; and (4) Abrasion resistance. The specified limits of these variables are given in Reference IV-8.

The average gross density of structural clay products is 123 pounds per cubic foot, with a variation from 104 pounds per cubic foot to 142 pounds per cubic foot. The compressive strength of bricks is usually measured with them lying flatwise (see ATSM C678C112), on the basis of their gross area. The compressive strength of load-bearing tile is also based on gross area, and that of floor tile on net area. A study made in 1929 of brick representing 37 percent of that year's output, indicated that the median compressive strength of the bricks tested were approximately 7000 psi. The ratio of the compressive strength measured lying on its edge to that measured with it lying flat is reported to vary from 0.74 to 2.3. The minimum value of the transverse strength or flexure strength of bricks is not specified and there is no direct relationship between it and the compressive strength. However, ASTM C67 includes a standard test for measuring this strength and reports it as the Modulus of Rupture. The modulus of rupture is obtained from the following equation:

$$MR = \frac{3\ WL}{2\ bd^2}$$

where: MR = Modulus of Rupture

W = Total load in pounds at point of breaking

L = Span between supports - inches

b = width of brick - inches

d = depth of brick - inches

The study referred to above also measured the modulus of rupture of bricks and found that the median was approximately 1200 psi, with slightly over five percent of the bricks tested falling below 450 psi and about the top seven percent between 2100 psi and 3450 psi.

The water absorption of clay bricks and tiles varies in an approximate inverse ratio to their compressive strength. Water absorption is determined by measuring the percentage increase in weight of a unit after any one of several treatments. The most common treatments are partially or completely immersing the unit in cold water for a period of time varying from a few minutes to several days or in boiling water for a period of one to five hours, or alternately applying vacuum and pressure to the immersed unit. Water absorption will vary with the method used. In no case, however, will it necessarily be a measure of the true porosity of the unit. In the study previously referred to, the median values of water absorption for the bricks tested were about nine percent for five-hour immersions in cold water, about ten percent for 48 hours immersion in cold water, and a little under 14 percent for five hours immersion in boiling water. Water absorption provides an indication of the probable relative durability of the materials when subject to weathering, especially when freezing and thawing conditions are expected.

Vitrified clay products generally have about the same resistance to abrasion as natural stone, depending on the raw materials and manufacturing process used and the degree of vitrification. A study conducted on the abrasion resistance of various bricks and its relationship to their compressive strength and water absorption is reported to have resulted in the following conclusions:

1. Abrasion resistance generally increases with compressive strength in such a way that a plot of the relationship can be closely approximated by a hyperbolic curve.
2. Whether the unit is dry or saturated has very little effect on its abrasion resistance.

In order to determine the economics of using commercially available structural clay bricks or tiles in riverbank stabilizations, a standard unit was chosen which offers sufficient durability, strength and abrasion resistance, is readily available from most manufacturers, and is relatively inexpensive. In consultation with representatives of the Structural Clay Products Institute, it was decided that a standard floor tile, four inches thick, twelve inches long, and twelve inches wide, with three cells running through one 12-inch dimension, would probably be the unit which best satisfied these requirements. These units are available in the Mississippi State area at a cost of approximately \$0.16 per tile. These units could be easily strung on wires to form mattresses because of their cellular construction. A comparison of the cost of these and other ceramic materials given consideration is presented in Table I-3. The shapes of some of the other available units are illustrated in Reference IV-10.

Poreen: Poreen is a new ceramic material developed in Germany and recently introduced into this country by Struthers Scientific and International Corporation (Ref. IV-12). The major ingredients of the material are sand (or other suitable filler), lime, and water, with a small portion of cement. The solid materials are ground very fine and scientifically combined and fused to give a product with very uniform properties and high density and strength. The materials are fused through wiring in an autoclave where they are subjected to a four or five hour heating cycle with saturated steam at a pressure of 235 psi and at a temperature of about 400 degrees Fahrenheit. The finished product has a specific gravity of 1.9 to 2.2, a compressive strength ranging from 9000 to 20,000 psi, and a water absorption of 12 percent (Ref. IV-2).

Commercially, this product is initially being proposed for use as a floor or facing tile because of its excellent weathering properties and abrasion resistance, and its uniformity. No plants have as yet been built near the Lower Mississippi. However, the promoter believes that one could easily be built which would use the river sand as raw material and would produce the high quality bricks mentioned above at a cost of approximately $15.00 per ton. If a somewhat lower quality product was desired for use in the Mississippi River bank protection, the cost would be less. It could be made with holes for stringing and could be of any desired size. On the basis of $15.00 per ton, and assuming that a unit 3 inches thick, 12 inches wide, and 24 inches long would be satisfactory, the brick cost is computed to be about $0.25 per square foot of protection. This is compared to the cost of other types of ceramics in Table I-3.

Proposed Pyroplastic Method of Forming Ceramics

Current conventional methods of forming and firing clay products for structural use involves the extrusion or cold pressing of the raw clay into the desired shape followed by drying and the exposure to temperatures between 1800 degrees and 2400 degrees Fahrenheit. The purpose of the firing is to form a glassy bond to the crystalline material and thus develop a high-strength, weather-resistant and sometimes impervious finished product. These processes require equipment for grinding and sizing the raw material, mixing it with the proper amount of water, and forming the desired shape. The kilns used for firing are large, permanent structures and the firing process requires from two to five days per unit, depending upon the type kiln used. Pyroplastic forming utilizes the simultaneous application of heat and pressure to form the glassy bond and form the desired shape.

Heating: The raw clay would be heated to a temperature of from 1800 degrees to 2200 degrees, depending upon the characteristics of the clay. There are three methods of heating the clay being considered:

1. Rotary Kiln - This is a refractory-lined cylindrical shell supported in a near horizontal position. The tube rotates and the material passes through the kiln counter-current to the flow of heat. The capacity and rate at discharge is determined by not only the diameter and length, but also by the angle of pitch and the rotational speed.

2. Moving Grate Sintering Machine - This type of equipment has been used quite commonly for the production of light-weight aggregate and for the roasting of metallic ores. A combustible material (usually coal) is mixed with the substance and placed upon a moving grate. As the grate passes under the combustion area, the coal ignites furnishing sufficient heat to maintain combustion on the grate. In this way the heat is used more efficiently and the finished product comes off the grate in a more controlled manner. This equipment could be utilized as a means of heating the clay by extending the combustion zone over much of the length of the grate. The heated clay would then be diverted into the presses as needed.

3. Fluidized Bed Calcining Unit - A batch prototype of this type of equipment has been designed and tested independently of this work at the University of Washington. The basis of the operation is to use hot combustion gases as the fluidizing medium eliminating the need of bulk refractories and simplifying transport of the hot material. Initial testing indicated that a clay-type material could be adequately utilized and that four pounds of material could be heated to 1800 degrees Fahrenheit in three minutes using 200 cubic feet per minute of an air-gas mixture. Since there are no further data than this on the process, it is practically impossible to give a realistic engineering cost evaluation. The clay would be fed into the unit in a dry state and would be carried on a fluidized bed through a heating portion where the temperature would be raised to the desired value. The heated clay would then be transported to the forming locations.

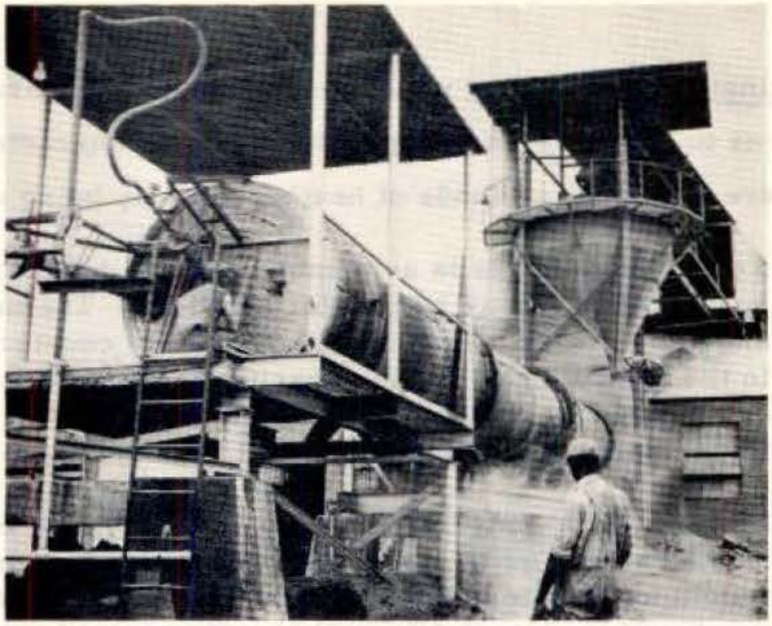

Fig. IV-1. Rotary kiln.

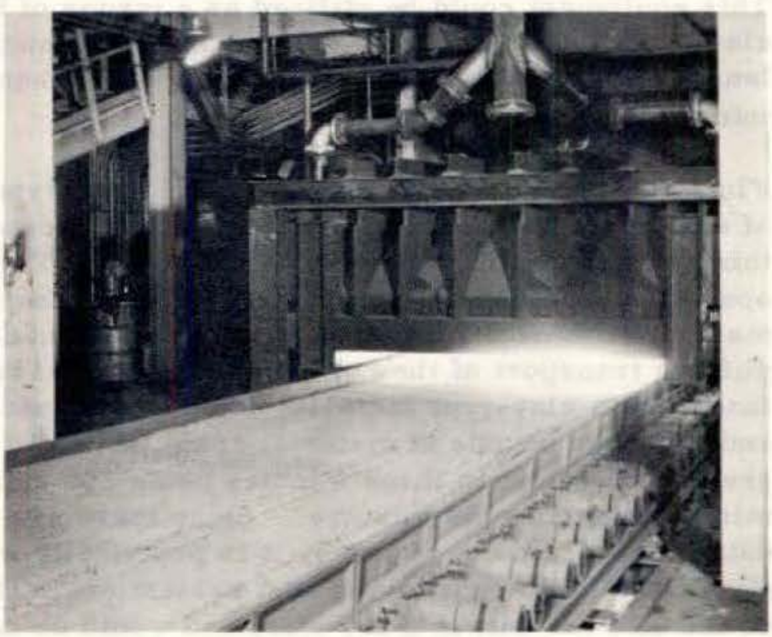

Fig. IV-2. Moving grate sintering machine.

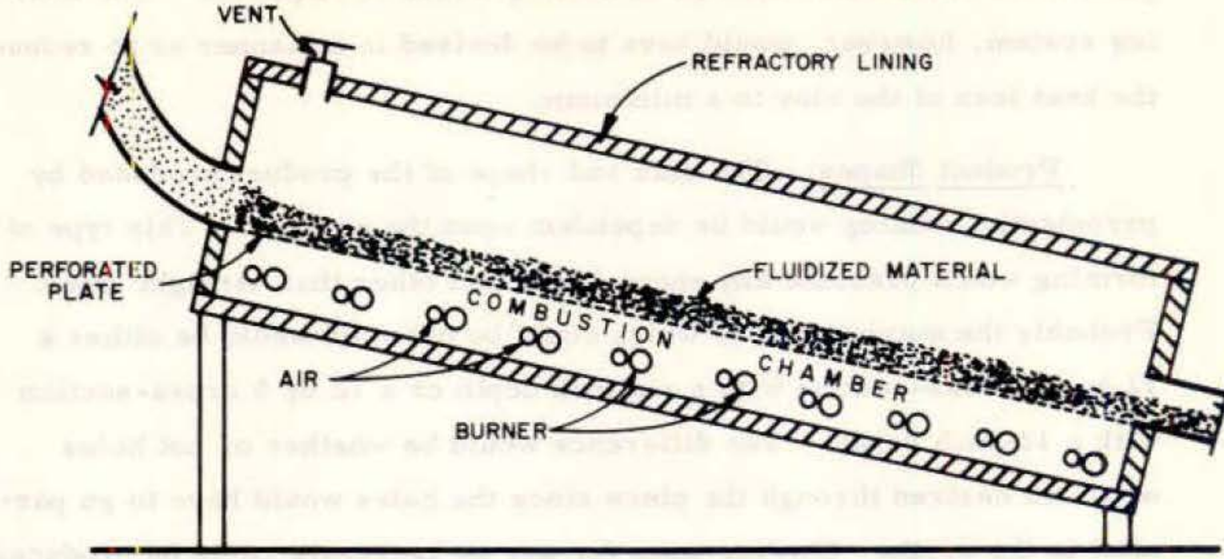

Figure IV-3. FLUIDIZED BED CALCINING UNIT

Pressing: The heated clay would be appropriately transported to pneumatically operated presses, similar to the concrete block press but with greater pressures available. These presses would be designed to press a unit 12 inches by 6 inches cross-section with a depth of 12 inches. The heated clay would drop into the die cavity in such a manner as to keep the heat loss to a minimum and the pressing cycle would begin. It is estimated that one unit could be pressed every eight to ten seconds. After pressing, the formed unit would pass by belt conveyor through an annealing oven to relieve any stresses set up in the glasses during forming. The temperature of this should be relatively low (800 to 1000 degrees Fahrenheit). It should be explained here that refractory metal dies (Inconel, Hastelloy, etc.) would be required to withstand the temperatures of the hot clay. These would be expensive due not only to the material cost but also the machining and die forming costs. The final design would be dependent upon experimentation during pilot plant tests since some method of cooling would be required. The cooling system, however, would have to be devised in a manner as to reduce the heat loss of the clay to a minimum.

Product Shapes: The size and shape of the product produced by pyroplastic forming would be dependent upon the end use. This type of forming would preclude any shape which had other than straight sides. Probably the maximum size which could be obtained would be either a 12 by 12 cross-section with a six-inch depth or a 12 by 6 cross-section with a 12-inch depth. The difference would be whether or not holes would be desired through the piece since the holes would have to go parallel to the depth. Similar units for use as aggregate could be produced making the aggregate size any dimension desirable. It should be pointed out, however, that additional costs in die preparation would be incurred in the latter instance.

Utilization: It is envisaged that products prepared in this manner could be used as aggregate, riprap or as units for post stressed members. It is felt that a method similar to the post stressing of beams and slabs might be used as a replacement of the articulated concrete mattresses. In any case, one of the principal advantages of the proposed method is that it requires a minimum amount of equipment to produce the desired shapes, and the entire operation could be barge-mounted.

Cost Estimates: Development of cost estimates is difficult since no prototype or pilot plant equipment was available. The estimates, therefore, are based upon comparable equipment with the costs modified to take into account the uniqueness of the pyroplastic forming techniques.

It is estimated that one press will be able to produce the equivalent of five cored 12 inches by 12 inches by six inches per minute, each weighing approximately 25 pounds. Blocks without coring of the same size would weigh approximately 70 pounds. The former could be utilized as a substitute for concrete mattresses, whereas the latter would be utilized for riprap. It is anticipated that one large rotary kiln capable of producing 360 tons per day would feed four presses. The cost figures below take into account (1) labor of three men per shift with one general supervisor, (2) availability of gas as fuel, (3) depreciation of equipment based upon a five-year linear depreciation program, and (4) die replacement costs. It is contemplated that the material used will be that which is normally removed from the bank into the river and no material costs were included. It is felt this basis is justified by the fact that this work will have to be done regardless of the type of bank stabilization used. The cost estimates below do not include any overhead.

Type of Product	Cost per ton
Mattress Blocks	\$2.60
Riprap	4.70
Aggregate	2.60

These costs are based upon a utilization of a three-shift crew working approximately 20 hours per day and 200 days per year. The riprap cost per ton is considerably higher than the cored mattress block cost, because it would take longer to form the solid riprap blocks than it would the cored mattress blocks. As the distance from a free surface to the center of a ceramic material increases, it becomes more difficult to achieve the required combination of heat and pressure at the center.

The estimated cost of capital equipment other than the barge or barges upon which it might be installed is approximately \$300,000. This includes a rotary kiln, four presses, an annealing furance, the materials handling equipment, air compressors, and compressed air storage.

These cost figures are believed to be adequate for comparative purposes only. The cost figure for aggregate, however, may be as much as ten to twenty percent higher than the actual cost.

PART IV

Chapter 2

SYNTHETIC SHEETING AND FABRIC REVETMENTS

General

The material classification of synthetic sheetings and fabrics includes all those materials normally designated as plastics or synthetic elastomers. The designation of plastics is generally accepted as applying to "certain organic substances, ... mostly synthetic condensation or polymerization products." An elastomer is more vaguely defined as "A material, usually synthetic, having elastic properties akin to those of rubber." Synthetic is defined as "Artificial, not derived immediately from a natural product" (Ref. B-6).

These definitions are necessarily vague, and there is no distinct division between synthetics and non-synthetics. However, in the plastics industry common usage assigns materials to one of three general classifications: synthetic materials, semi-synthetic materials, and natural materials. Examples of synthetic materials will be given in this chapter. Some materials which are usually included in the other two classifications are as follows:

Semi-Synthetic Films - cellulose and its derivatives

Semi-Synthetic Fibers - rayon and cellulose acetate

Natural Films - paper

Natural Fibers, Organic - cotton, wool, jute

Natural Fibers, Inorganic - asbestos

In this chapter none of the natural or semi-synthetic materials are discussed. Almost all of the natural and most of the semi-synthetic sheeting and fibers do not have the strength, durability, or resistance to water which is required of any material to be used in riverbank stabilization.

Synthetic sheeting has been used extensively in this country for water barriers in structures, reservoirs, and canals. Synthetic fabrics have been used in Europe in other hydraulic engineering applications. Pertinent information on these installations and the attendant testing are given in this chapter and in Chapter 4 of this part. Previous utilization of these materials in applications similar to riverbank revetments is discussed under "Previous Applications" immediately following this introduction. The general physical properties of these materials and other factors which should be considered by a Civil Engineer when evaluating the feasibility of their utilization in any such application are discussed under the following section, "General Properties of Synthetic Sheeting and Fabrics." Specific properties of some of the films and fibers investigated during this study are discussed under the following two sections, "Synthetic Films" and "Synthetic Fibers." Some of the methods of utilizing these materials for riverbank protection are discussed in the last section of this chapter, "Proposed Applications and Placement Methods."

Previous Applications

The Dutch and the Germans have been most active in experimenting with synthetic fabrics in erosion control and other hydraulic engineering applications. Some of the installations in which the Dutch are reported to have used such materials are as follows:

1. Sunk sheets
2. Sand bags
3. Revetment mattresses
4. Filters between concrete blocks
5. Seepage control membranes
6. Filters for toe drains on dikes
7. Protection for the toe of canal banks

A polyamide (nylon 6) fabric manufactured by Alqemene Kunstzude Unie N. V. -Arnhem (trade name "Enkalon") was used in all of the above applications. Those of greatest interest to this study are numbers 1, 2, and 3, and perhaps number 4. The use of synthetic sand bags is discussed in Chapter 4 of this part. The others are discussed below.

The Dutch have used Enkalon mattresses in protecting the sea bed in front of dikes and the upstream point of an island in the Magdalena River in Colombia against erosion (Ref. IV-25, 28, 29, 30, 31, 32). In the former case the mattress was a double sheet of Enkalon sewn together at regular intervals to form a series of tubes. These tubes were filled by injection with a sand slurry for ballast. The number of tubes filled was determined by the ballast weight required according to the force of the current and water in the area to be protected. The mattresses, with the required tubes already filled, were placed on a "pontoon" which laid the mattress as the pontoon was pulled along the sea bed in 50 to 65 feet depths of water.

The mattresses used on the Magdalena River (1961) were of essentially the same construction, but were placed in a different manner. In this installation 22 mattresses, 100 feet long by 64 feet wide, were placed on slopes as steep as 1:1 in depths of water up to 46 feet. The mattresses were placed from a barge in a manner very similar to the placing of the articulated concrete mattresses on the Mississippi River. Each mattress was fabricated from 12 strips of material approximately 100 feet long and 63 inches wide. Each strip was divided into six elements, approximately 16.4 feet long and 63 inches wide. Each element was composed of four cells tubes, approximately 15.8 inches wide and running the length of the elements. Thus when the full mattress was fabricated, it was composed of 48 longitudinal tubes, each divided into 16.4 feet sections. One end of the mattress was anchored to the top of the bank, and the remainder was rolled up on a drum on the barge. As the barge moved away from the bank, the mattress unrolled, and the tube sections were slit open, filled with sand slurry, and sewn shut. A heavy steel cylinder had to be placed on the mattress to keep it from being folded over by the river currents (up to ten feet per second).

Because of the experimental nature of this installation, the high cost of the imported materials and the large amount of hand labor required, the cost of protection was quite high, averaging $2.26 per square foot. The cost breakdown, as supplied by the owners, Shell Condor S.A., is given in Table IV-2.

Table IV-2

COST OF INSTALLING ENKALON REVETMENT MATTRESSES ON THE MAGDALENA RIVER, COLUMBIA, S. A.

Item	Equivalent Dollar Costs (U.S.)
1. Materials	$111,440
Other	
2. Salaries	532
3. Labor	11,116
4. River Craft	19,333
5. Workshops	4,507
6. Earthmoving Equipment	1,693
7. Preparation of Equipment	2,247
8. Expert Advice	14,598
Subtotal	$ 54,026
Total	$165,466

Protected Area = 6825 square meters = 7350 square feet.

Cost Per Square Foot

Materials	Other	Total
$1.52	$0.74	$2.26

To compute the final per square foot cost of fabric in the sand filled mattress, it is necessary to multiply the basic cost of a square foot of the material by a factor of approximately 2.74. This factor takes account of the facts that: (1) the mattress is fabricated from two thicknesses of material, and (2) after the mattress is filled with sand it is narrower than when it lies flat. On the Magdalena installations the original mattress was about 64 feet wide and the filled mattress was about 64 feet wide and the filled mattress was about 46.7 feet wide, resulting in a reduction factor of about 1.37. This, when multiplied by two to take account of the double thickness, results in the above factor of 2.74.

Although the Magdalena installation has not been in place for a sufficient length of time to judge its long term effectiveness, some preliminary conclusions can be reached. The first of these is that it apparently retains the fine riverbank sands effectively. A previous attempt had been made to stop the erosion of the island by using the Dutch Mat (Ref. II-133). This apparently was unsuccessful because the strong river currents sucked the sands out from beneath the mattress, causing it to sink out of sight. Shell Condor thus decided to experiment with the Enkalon mattresses. These have been effective in preventing scour because of their ability to retain the foundation materials and their flexibility which allows them to conform tightly to every slope irregularity.

The second preliminary conclusion is that materials such as Enkalon must be well protected against sunlight before they can be used above water. On the Magdalena River the mattress protection was started at the top of the bank. Difficulties with sunlight deterioration were anticipated, and an attempt to protect the Enkalon was made by spraying on a layer of RC-2 asphaltic cutback on the area to be exposed during low water. In addition, tube sections which were to be above low water were filled with an asphalt emulsion stabilized sand slurry to provide some resistance to abrasion if the Enkalon were damaged. Apparently the asphaltic cutback was peeled off by the flowing water, and within six months the exposed Enkalon had lost almost all of its initial strength.

Strips of Enkalon have also been used to connect concrete blocks into an articulated concrete mattress. The edges of these strips were embedded a few centimeters into adjacent blocks during casting. The bond between the fabric and the concrete is said to be sufficient to eliminate the need for any further reinforcing wire. It is not reported how large these mattresses are or how they were placed, but it is stated that the strength of the fabric was the controlling factor in determining the strength of the mattress. The fabric generally has a tensile strength of over 300 pounds per inch, which would be sufficient for the articulated concrete mattresses now used on the Mississippi.

General Properties of Synthetic Sheeting and Fabrics

Synthetic films and fibers are quite recent additions to the family of materials available for use in civil engineering construction. However, because of their many applications, the industry has grown at a very rapid rate over the past 20 or 30 years. This poses three major problems for an engineer attempting to choose which particular material will be best for any proposed application.

First, there are numerous basic types of these materials available. These basic types can be produced in innumerable combinations and formulations, depending upon the manufacturer and the specific physical properties desired. In addition, large scale research in the field is continuously producing more new types, formulations, and combinations, as well as new methods of production, resulting in a continuous influx of new materials with new properties.

The second problem is an outgrowth of the first. Because of the newness of the industry and its continuous influx of new materials, there is little data available to evaluate the long term properties of the materials in regard to resistance to water, sunlight, temperature, weathering, microbe attack, chemicals, etc. Thus, predictions of whether a particular material will perform satisfactorily for 25 to 50 years often must be extrapolated from the results of one to five years of experience, or less.

The third problem is that most of the physical properties of synthetic materials have been evaluated in comparison to paper and natural textiles which they often replace. This poses two difficulties. The first is that the terms are often unfamiliar to the Civil Engineer. The second is that many of the tests which are pertinent to the paper and natural textile fields are not as pertinent to civil design or construction.

Nevertheless, it was considered desirable to attempt to briefly summarize the properties of those materials found to be available which might be used in the protection of riverbanks against erosion. Two recent publications which present more detailed introductions to this subject are given as References IV-22 and IV-24.

There are certain characteristics of most synthetic materials, particularly plastics, which are very different from the characteristics of materials which the Civil Engineer customarily uses and which are pertinent to their application in riverbank stabilization. Although these characteristics will vary among individual formulations, they are sufficiently prevalent throughout this group of materials to warrant a general discussion. Variations of these and other characteristics among the different types of materials are discussed under the heading of "Synthetic Films" and "Synthetic Fibers."

One of the most significant of these general characteristics and one which is very important to the Civil Engineer, is the tendency of

Most plastics to suffer severe deterioration when subjected to actinic or ultraviolet radiation. Most plastics, when left exposed to strong sunlight, will substantially disintegrate within a period of time varying from a few months to a number of years. The rapid disintegration of the nylon mattresses used in Colombia has already been mentioned (page IV-24), and there are numerous instances in Civil Engineering and other fields of other plastics suffering similar deterioration. The difficulty of evaluating this tendency for any particular synthetic material is compounded by two of the problems mentioned above. The first is that normal applications of these materials do not entail the severe exposure that they receive under most Civil Engineering applications. Thus a typical plastics properties chart (Ref. IV-21, 22) will list the sunlight resistance of nylon, for instance, as good. The second is that many of the plastics and their compounds are so new that there are no long term data available on these tendencies even if such tests have been undertaken.

There are two general methods of protecting plastics against such disintegration. The first is based on the fact that this tendency will generally be greater in a transparent or translucent material than in an opaque material. Pigmentations, such as carbon black, can be included in the plastics formulation at the time of manufacture to render the material opaque and reduce this tendency. This procedure probably will not provide complete protection against deterioration. On the other hand, opaque materials will tend to absorb more heat which also has an adverse affect on the life of synthetics.

The second method of protection from sunlight attack is to shield the material by covering it. As mentioned above (page IV-24), this was attempted in the installation in Colombia. It was unsuccessful because the covering was apparently peeled off by the current. In Holland they have been investigating this problem and apparently have developed an epoxy plastic coating which is said to provide better adhesion and protection than the asphalt (personal communication).

They have also conducted some tests in Holland to determine the resistance of several different types of materials, especially spun yarns, to actinic deterioration (Ref. IV-17, III-52). Most of the samples included in these tests were polyamides because they seemed to be most suitable for use in sand bags. Some of the samples had black dyes such as logwood and Multamine Black B added to make them opaque. Others had chemical stabilizers added which tend to retard such deterioration. Others had nothing added to the formulation, but carbon black was added to the yarns during spinning. In addition, polyester, polyacrylonitrile, polyvinylidene chloride, polyethylene, linen, and glass fibers were also tested. Two of the diagrams included with the report on these tests are presented in Figure IV-4. Some of the conclusions can be summarized as follows:

1. The polyacrylonitrile, polyvinylidene chloride and polyethylene fibers seemed to demonstrate the greatest resistance of the unstabilized yarns.

2. Although several of the stabilizers added to the polyamide were effective, the best results were obtained with Nylon 6 (Enkalon) with "stabilizer 2" and carbon black.

3. Coarse thick fibers seemed to be considerably more resistant than fine thin fibers.

The need to protect the plastic films from sunlight deterioration is one of the reasons that such membranes are usually buried when used in lining canals and reservoirs. The Bureau of Reclamation has been conducting outdoor exposure tests on a number of different films, and plans to publish their results to date sometime during 1964.

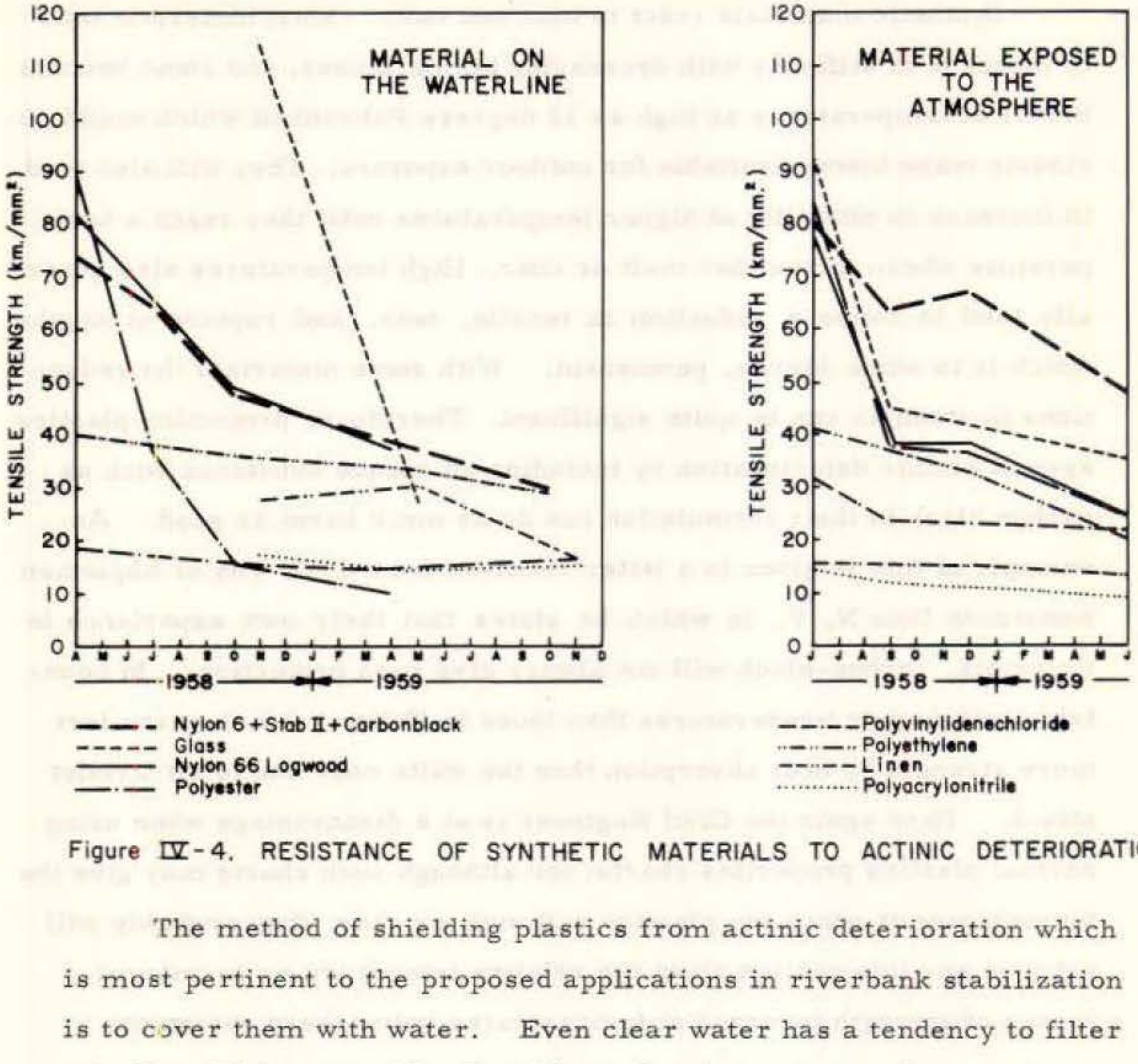

Figure IV-4. RESISTANCE OF SYNTHETIC MATERIALS TO ACTINIC DETERIORATION

The method of shielding plastics from actinic deterioration which is most pertinent to the proposed applications in riverbank stabilization is to cover them with water. Even clear water has a tendency to filter out ultraviolet rays. The Dutch have found that ten centimeters (approximately four inches) of water is sufficient to filter out most of the destructive rays. In the muddy waters found in the Lower Mississippi probably even less would suffice. Experimental installations of plastic materials under water have experienced very little loss in strength. Therefore, it can be predicted that any such materials utilized below low water in riverbank stabilization would probably retain sufficient strength during the desired life of the installation to maintain their effectiveness.

Synthetic materials react to heat and cold. Most materials tend to increase in stiffness with decreasing temperatures, and some become brittle at temperatures as high as 32 degrees Fahrenheit which would obviously make them unsuitable for outdoor exposure. They will also tend to increase in pliability at higher temperatures until they reach a temperature where they either melt or char. High temperatures also generally tend to cause a reduction in tensile, tear, and rupture strengths which is to some degree, permanent. With some materials the reductions in strength can be quite significant. Therefore, protecting plastics against actinic deterioration by including an opaque substance such as carbon black in their formulation can do as much harm as good. An example of this is given in a letter received from J. G. Vos of Alquemen Kunstzude Unie N. V. in which he states that their own experience in Colombia, carbon-black will not always give good protection. In countries with higher temperatures than those in Holland, black yarns lost more strength by heat absorption than the white ones due to ultraviolet attack. Here again the Civil Engineer is at a disadvantage when using normal plastics properties charts; for although such charts may give the temperature at which the plastics will melt or char, they probably will not give any information about the relative temporary or permanent losses of strength incurred at temperatures below these extremes.

Synthetic materials have a tendency to dissolve in or to absorb water. Those plastics which have been specially formulated to dissolve in water need not, of course, be considered. However, many synthetic materials, especially when in the form of multi-filament yarns, will tend to absorb some water and, as a result, lose some strength. This water absorption and resulting strength loss may be a process which will take place for a short period after the material is first submerged and will then cease, or it may be a process that will continue until the material has lost all of its strength. Here again, the normal plastics properties charts can be misleading to the Civil Engineer, for a rating of a particular plastic's resistance to water is more apt to be an indication of the initial amount of water absorption and strength loss than an indication of what will occur over a long period of time.

Synthetic materials can suffer severe deterioration from attack by micro-organisms and animals. Existing knowledge is insufficient to predict when this may happen. Thus the material should be tested under the specific anticipated conditions. Tests conducted by the Bureau of Reclamation in cooperation with the Agricultural Research Service and Utah State University (Ref. III-2, IV-14) have subjected various plastics to bacterial attack by burying them in compost piles. The results of these tests to date (1962) have been summarized as follows (Ref. III-2):

> "From these tests and those of the Bureau, and from field observations, it has been found that the films are highly resistant to bacteriological deterioration and that the tensile strength and flexibility of plastics exposed to ten-year composted soil burial were virtually unaffected."

They have also concluded that plastics reinforced with "such organic materials as paper, canvas, and burlap, are not desirable because the organic type fabrics are subject to micro-organism attack." They are planning to publish a technical memorandum during 1964 which will give more up-to-date results in detail. An example of an unreinforced plastic which was destroyed by micro-organism attack, however, was reported in a discussion with a representative of Seiberling Rubber Company. A test installation of their polyurethane film as a reservoir liner was made and revealed that within a year, apparently because of such attacks, the film had completely deteriorated.

Synthetic materials are generally less resistant to abrasion, puncture, and tearing than most of the materials with which the Civil Engineer is accustomed to working. Here again, most comparisons made within the plastics industry are in relation to such materials as paper and natural textiles and thus may prove misleading. These materials when compared to metals, concrete and other similar materials will obviously rate very low in all three respects. However, when compared to other synthetic materials, paper or textiles, they will probably rate very well, and some will be much better than others.

Abrasion resistance, if the definition of abrasion is restricted to the attrition resulting from the impact of sand and silt carried by the water, will probably not be a serious problem affecting the use of these materials in riverbank stabilization. Tests have indicated that, within limits, a flexible pliable material is apt to be more resistant to this type of abrasion than one that is more rigid (Ref. III-48). The flexible materials absorb the force of the impact by deforming, whereas the rigid materials are worn away. However, if abrasion is taken in a wider sense to include all attrition whether resulting from the above causes or the friction induced by contact with other rough surfaced large solids, then most of the synthetic materials will rate poorly. Being relatively soft they are not very resistant to friction.

Puncture resistance is a concept which is probably unfamiliar to the Civil Engineer accustomed to working with metals and concrete. However, when dealing with films and fabrics it is a very definite consideration. The resistance of any film to puncture is primarily a function of three variables, the sharpness of the object inducing puncture, the tensile strength of the material, and the ability of the material to elongate before rupture. Probably no thin film, unless it is reinforced with wire, will offer significant resistance to puncture by sharp objects. However, such films can offer high resistance to puncture by blunt objects depending upon the film's tensile strength and stretching ability. The ability to elongate before rupture is important because it allows the foundation materials to absorb some or most of the force of the object.

The tearing strengths of synthetic films or fabrics is an important consideration when evaluating their suitability for any Civil Engineering application such as riverbank stabilization. Tearing strengths usually represent the amount of force required to continue tearing a film after starting. It is usually measured by the Elmendorf Test and expressed in grams (for a one mil thickness) or by SPI-ASTM D 1004 and expressed in pounds per inch (of thickness). The force required to start the tear will be higher (up to 60 times) than that required to maintain it, and will depend upon the jaggedness of the starting point. Tearing can also be restricted by reinforcing the material.

Plastics are generally divided into two major classifications, thermosets and thermoplastics. The definitions of these two classifications are (Ref. B-6):

> Thermosetting - "Compositions in which a chemical reaction takes place while they are being molded under heat and pressure; the appearance and chemical and physical properties are entirely changed, and the product is resistant to further applications of heat (up to a charring point)."
>
> Thermoplastic - "Becoming plastic on being heated. Specifically any resin which can be melted by heat and then cooled, the process being able to be repeated any number of times without appreciable change in properties."

This distinction can perhaps be illustrated by analogy with two materials which are not plastics -- coal and glass. Coal could be considered similar to a thermosetting composition in that, although perhaps originally formed under condition of heat and pressure, it cannot be melted or heated to high temperatures without changing its chemical composition and physical properties. Ordinary glass, on the other hand, could be considered similar to a thermoplastic in that it can be heated up and melted any number of times, and changed into different forms, without changing its inherent chemical and physical properties.

Most plastic films and fibers are made from thermoplastic materials. Many elastomers, however, have more nearly the properties of thermosets. Films made from thermoplastic materials have inherent advantages over those made from thermosetting materials when considered for riverbank stabilization. The first of these is that the material can usually be sealed or jointed by application of heat rather than requiring adhesives or solvents. For this reason thermoplastics can usually be sealed or jointed in less time, with less bother, and for less cost than thermosets. Such heat sealing can be accomplished either by applying heat directly (e.g., by a heated iron or bar) or, more effectively, by inducing heat in the material through high frequency generators, etc. The second advantage is that any perforations which must be made in the material to allow water drainage can be made through heat puncture rather than mechanical puncture. Perforations made in films by mechanical puncture are apt to have jagged edges which could easily develop into tear lines. Perforations made in thermoplastic materials by heat puncture, however, will have an extra layer of plastic deposited on their peripheries. This will tend to act like a reinforcing grommet in preventing the development of tear lines from the perforation.

The third advantage of thermoplastics over thermosets is one which may have more significance in the future. This is in regard to the relatively new procedure of extruding plastic netting (Ref. IV-16). Virtually any thermoplastic can be used in producing such netting. The netting can be produced with practically any size openings, from a very open mesh to one so close that it is almost impermeable. It can also be produced with various strand thicknesses. Most of the existing machinery has been developed in England or France and is capable of producing netting in tubular form, or in sheets up to 20 feet wide. At the end of 1963 four companies in the United States (DuPont, Nalle Plastics, Inc., Rexall Drug & Chemical, and Union Carbide) had begun producing such netting on license from the European firms. It is quite possible that the cost of this extruded netting may be reduced in the future below that of woven fabrics. The Union Carbide Plastics Company estimates that a netting produced from an inexpensive plastic such as polyethylene, with a sufficiently small mesh size to retain sand, and with a strand thickness of about 1/16 inch, would cost approximately ten cents per square foot (personal communication).

Synthetic Films

In the manufacture of plastic films there are four main items which affect their chemical or physical properties. Simons & Church (Ref. IV-24) list these as follows:

1. The type of plastics resin used to make the film.
2. The compounding ingredients used with the basic resin.
3. The method by which the film is produced.
4. The thickness of the film (in this discussion the term "film" is used in its general sense to include all

continuous flat sheets of material up to about one-half inch in thickness, contrary to the more restricted usage adopted by Simons & Church and prevalent throughout the plastics industry which limits "films" to thicknesses of ten mils (0.010 inches) and applies the term "sheeting" to all greater thicknesses).

In addition to these four main items, the physical properties of the film can be further modified after its formation by such techniques as molecular orientation, variable rates of cooling, stress releasing, laminating, and reinforcing. Molecular orientation is produced by stretching the film after it is produced. It can be stretched in one direction, or uniaxially oriented; or it can be stretched in both directions, or biaxially oriented. Such orientation tends to increase the tensile strength of the material in the direction of stretching, but concurrently, it tends to reduce the tearing resistance of the film parallel to orientation. Internal stresses in the plastic film can also be induced and controlled by varying the rate of cooling; and, if induced, can be relieved by reheating the material after production. Thermoplastic films can also be easily reinforced with other fibers to increase the tensile strength and/or tearing resistance. The reinforcing is usually accomplished by thermally laminating individual reinforcing strands or a woven fabric between two separate plastic films. Typical fibers used in reinforcing plastics are glass, nylon, polyesters, and polyvinyl chloride. One company engaged in producing reinforced plastics by a patented process for the construction industry is the Griffolyn Co., Inc. of Houston, Texas.

The first problem in choosing which synthetic materials might be suitable for use in the methods of riverbank stabilization under consideration was to eliminate those which obviously were not suitable. The criteria which were used to eliminate those which would be unsuitable are as follows:

1. The material must not deteriorate when submerged in water.

2. The material must have sufficient flexibility or pliability to allow it to conform to variations in the bank slope.

3. The material must not have a very low tensile or tear strength.

4. The material must retain all desirable characteristics within a temperature range from 0 degree Fahrenheit to 140 degrees Fahrenheit.

Based on available information the following materials as listed in the "Films Chart" presented in Reference IV-21 were thus eliminated from further considerations:

Styrene and styrene compounds

Cellulose and cellulose compounds

Polymethyl Methacrylate

Polycarbonate

Polyvinyl alcohol

Vinyl-nitrile rubber

Those materials which did satisfy these basic criteria and were studied in greater detail are listed in Table IV-3 (page IV-45) with their pertinent physical properties and approximate costs. The tests used in determining these physical properties are listed and briefly described on Sheet 2 of Table IV-3. The following is a short description of each synthetic film.

Butyl Rubber: Butyl rubber is a synthetic rubber which is superior to natural rubber for many civil engineering applications. Its physical properties are superior to most plastic films in many regards. It suffers very little deterioration through ultraviolet attack, weathering and aging, in contrast to many plastics. It is also very resistant to micro-organism attacks, soil chemical attack, and root penetration. It is said to have approximately one-half the permeability of polyethylene and one-sixtieth that of polyvinyl chloride. Field durability tests conducted on this material in the Western United States have shown that it loses almost no strength after 10 to 15 years of exposure or burial (Ref. IV-15). As indicated in Table IV-3, it has an elongation of 500 percent at rupture and an initial tearing strength (ASTM D-412-61T) of 200 psi. It is normally supplied in thickness of 1/32, 1/16, and 3/32 of an inch, but it can be made either thinner or thicker. The thinner sheets are not normally supplied because existing production methods do not allow sufficient control of thickness variations to ensure that pin holes do not occur. Such pin holes, however, would not be detrimental to possible applications in riverbank stabilization, and thus the possibility of using the thinner and less expensive sheets might be considered.

Since butyl rubber has the characteristics of a thermosetting material, perforations for seepage would have to be made by mechanical puncture. Bonding and jointing are usually accomplished with adhesives, although sewing might be possible for the thicker sheeting.

Fluorocarbons: Fluorocarbon sheeting (e.g., DuPont's "Teflon," Minnesota Mining and Manufacturing's "TFCE," and Allied Chemical's "Aclar") is a relatively new material which is especially noted for its toughness, its lack of friction, and its high resistance to chemical attack, heat and cold. It is a very dense material with fair flexibility and pliability. It is highly resistant to actinic deterioration and weathering. Tests to evaluate its weathering resistance have included seven year's exposure in Florida, 5000 hours in a weatherometer, and 6100 hours exposure to high intensity ultraviolet light at 50 percent relative humidity and temperatures from 130 degrees Fahrenheit to 140 degrees Fahrenheit, all with no measurable degree of deterioration (Ref. IV-21 p. 480).

Since this material is a thermoplastic, it could be heat sealed and perforated by heat puncture. It is one of the most expensive sheetings available, costing a little over a dollar per square foot for a ten mil thickness. As has been the case with many other films, this cost may be reduced in the future through improved production methods.

Nylon 6 (Polyamide): Nylon is available in five different formulations -- nylon 6/6, nylon 6, nylon 11, nylon 6/10, and nylon 8 -- the first two of which are the only ones now normally fabricated as sheeting. Both are characterized by high strengths and high tearing resistance. They also resist abrasion well and are not easily damaged by sharp objects. They are relatively stiff films but have a high folding endurance. Some nylon films have a low resistance to water, others are much better. All have a relatively high water absorption and are not particularly resistant to sunlight deterioration. They cost approximately $0.13 per square foot for a ten mil thickness.

Being thermoplastics, nylon films could be perforated by heat puncture and could be heat sealed.

Petroleum-Resin-Rubber Compound: The Standard Oil Company of New Jersey has very recently developed a new petroleum-resin-rubber elastomer sheeting which they plan to market for lining canals and reservoirs.

They plan to produce it in a one-eighth inch thickness. This material has neither gone into production nor been officially named yet. Because the material has thus far only been produced in the laboratory, no information is available on its long term resistance to water, sunlight, or general weathering deterioration. However, the manufacturer predicts that it will be very good in these respects. Laboratory produced samples have tensile strengths of about 250 pounds per inch for an one-eighth inch thick specimen. Bonding methods have not been definitely established but it is known that solvents and adhesives will work, and it is predicted that heat sealing can also be used. It is predicted that the material will cost from three to five cents per square foot.

Polyesters: Polyester sheeting (such as DuPont's "Mylar," Minnesota Mining and Manufacturing's "Scotchpak," Eastman Chemical Products "Kodar," and Goodyear Tire and Rubber Company's "Videne") is also a relatively new material. It is said to be the toughest and strongest of the plastic films and, although relatively unpliable, has a very high folding endurance, tensile strength and tear strength. These films are usually bi-axially oriented during production so that their strength properties are maximized. They have a high dimensional stability and are very resistant to attack by most chemicals and water. They have a medium to excellent resistance to ultraviolet deterioration and weathering. They are thermoplastics and thus can be perforated by heat puncture and heat sealed. They cost approximately \$0.10 per square foot in 10 mil thickness.

Polyethylenes: Polyethylene films can be produced in low, medium, or high densities. Tensile strength generally increases with higher densities, but pliability, tearing strength and the percent elongation at rupture are apt to decrease. The quantity of polyethylene films produced is second only to cellophane among the plastics. It is used extensively in packagings as well as in such civil engineering applications as moisture barrier for buildings and canal and reservoir liners. It has excellent resistance to water but only fair to good resistance to sunlight and weathering.

Being a thermoplastic it can be heat punctured and heat sealed, though not as readily as some of the other thermoplastics. It has a very low cost, however, varying from $0.012 per square foot for low density film to $0.019 per square foot for medium density film, for a 10 mil thickness.

Polyethylene-Vinyl-Acetate Copolymer: This is a new film produced by the Union Carbide Company primarily for Civil Engineering applications such as canal and reservoir liners. In its formulation an attempt has been made to combine the advantages of low density Polyethylene, such as low cost and high elongation, with those of the vinyls, such as higher strengths and pliability. Because of the relatively high pliability and stretching ability, the manufacturer considers the material highly puncture resistant. Although the newness of the material precludes any information being available concerning the long term resistance of the material to water, sunlight, or weathering, it should be excellent in regard to water and, with the addition of carbon black to its formulation, is predicted to have a life of 20 or more years when exposed to sunlight and weathering.

In thicknesses up to 10 mils it can be produced in single sheets up to 40 feet wide on continuous tubing up to 20 feet wide when lying flat. In thicknesses from 10 to 40 mils it can be produced in sheets about 48 inches in width.

Being a thermoplastic, it can readily be heat sealed and perforated through heat puncture. The basic cost for a 10 mil thickness is predicted to be $0.02 per square foot, and the manufacturer says that perforations and reinforcing, if desired, should add an additional two to three-tenths of a cent per square foot.

Polypropylene: Polypropylene sheeting is a relatively recent addition to the family of plastic films available in the United States, being first introduced in 1958. Although of a higher cost than polyethylene, it has several properties which make it superior to polyethylene in some applications. It has a relatively high strength and tearing resistance (when unoriented), but it is also relatively stiff and has a low elongation at rupture. Much of the film produced has been either uni-axially or bi-axially oriented which increases its tensile strength, resistance to abrasion, and stiffness, but greatly decreases its tearing resistance.

It is also suitable for heat sealing and perforating by heat puncture. Although this film is not normally available in the thicknesses over three mills, it has been estimated that a 10 mil thickness would cost about \$0.036 per square foot.

Polyurethane: Polyurethane is one of the most wear and abrasion resistant plastics available. It has a relatively high tensile and tear strength combined with good elongation before rupture, and it can be quite pliable. However, it tends to absorb a fair amount of water and lose some strength when submerged (which is the reason that its resistance to water is only given as "good" in Table IV-3). The major difficulty with this sheeting is its apparent susceptibility to attack by micro-organisms. An attempt to use it as a reservoir liner resulted in a rapid failure which was ascribed to bacterial attack (page IV-31). Seiberling Rubber Company predicts that this tendency could probably be eliminated by the addition of certain anti-microbe inhibitors to the formulation, but no experiments have apparently been made with such a modified film to determine either the success of an inhibitor or its effect on the physical properties of the plastic.

This material can also be heat sealed and perforated through heat puncture. It costs approximately $0.13 per square foot for a 10 mil thickness.

Polyvinyl Chloride: Polyvinyl Chloride (or PVC) is the most common film used for lining reservoirs and canals. In its pure form PVC is a strong rigid material, but in its formulation certain "plasticizers" can be added which give the film great flexibility and pliability. These plasticizers are essentially solvents which partially dissolve the film, interrupting the rigid molecular structure. The more plasticizer added to the formulation, the more pliable is the film. However, the plasticizers are also the part of the formulation which is most susceptible to deterioration by chemical, microbe, and sunlight attack, and, if the film is left exposed, they are apt to be leached out within a year, leaving a weak brittle residue. For this reason it is especially important that PVC films be protected.

High pliability combined with moderate tensile strength, low water absorption, good tear strength, and fair elongation at rupture are the properties which have made it attractive for lining reservoirs and canals. It is a thermoplastic, and several of the more recent reservoir installations have utilized heat sealing methods. Its cost is approximately $0.05 per square foot for a 10 mil thickness.

Polyvinyl Fluoride: Polyvinyl Fluoride Films (e.g., DuPont's "Tedlar") are fairly expensive and are only produced in thicknesses up to two mils at this time, although they could be produced in greater thickness if required. The outstanding characteristics of this material are its excellent resistance to sunlight deterioration and its high abrasion resistance. Film samples have been exposed in Florida for ten years and have suffered little deterioration. The material is predicted to have a 25-year life under normal outdoor conditions when used, (without pigmentation) as a surface protection for wall panels, etc. It is produced in several formulations which vary from one with a high tensile strength (17,000 psi) and high flexural strength to one having a lower strength (8000 psi) but higher tear resistance and pliability. It has a relatively low elongation before rupture.

The film thicknesses now available discourage sealing by heat, but it is a thermoplastic. A 10 mil thickness is estimated to cost approximately $0.30 per square foot based upon the basic per pound cost of the resin.

Vinylidene Chloride-Vinyl Chloride Copolymer: This film (e.g., Dow's "Saran") has many of the characteristics of Polyvinyl Chloride, except that it is more impermeable to water and gas and is more chemical resistant. It also requires the addition of plasticizers to its formulation to make it pliable, with the resulting disadvantages therefrom. It is a thermoplastic and can be heat punctured or heat sealed. The film costs approximately $0.094 per square foot for a 10 mil thickness.

SL-64-22-1: This is a new plastic film under development by Monsanto Chemical Company which is supposed to have properties similar to polyvinyl chloride at a lower cost. Its pliability and puncture resistance is said to be good, and its resistance to ultraviolet deterioration very good. The estimated cost is approximately $0.03 per square foot.

Synthetic Fibers

Plastics, of course, can also be produced in the form of fibers which can be woven into fabrics. Fibers can be either monofilament

Table IV-3

PROPERTIES OF SYNTHETIC FILMS [1]

No.	Name	Specific Gravity	Tensile Strength psi [2]	Tearing Strength gms/mil [3]	Tearing Strength lbs/in. [4]	Bursting Strength 1 mil [5] Mullen Points	Elongation of Rupture [2] %	Water Absorption in 24 hrs. % [6]	Resistance to Water	Resistance to Sunlight	Resistance to Heat °F [7]	Resistance to Cold °F [7]	Sealing Methods	Approximate Cost for 10 mil th. per sq. ft.
1.	Butyl Rubber	1.20	1,400- 1,800		(200) [8]		500	Nil	E	E	200-250	- 30	Adhesive	$0.10 [9]
2.	Fluorocarbons	2.15	2,500- 3,000	125		11	300	Nil	E	E	400-525	-425	Adhesive Heat	$1.03
3.	Nylon 6	1.15	7,000-10,000	30- 210	1000	Elongates	200-250	9.5	E to P	F-G	180-375	-100	Heat Adhesive	$0.13
4.	Petroleum-Resin-Rubber		(2,000)						(E)	(E)			Solvent Adhesive Heat	$0.03-$0.05 [10]
5.	Polyesters	1.38-1.395	23,000-40,000	12-40	650-1740	30-55	50-100	0.8 [11]	E	M-E	300	-140	Heat Adhesive	$0.10
6.	Polyethylene - low density	0.910-0.925	1,250-2,500	50-300	66-575	10-12	200-800	0.01	E	F-G	200	- 70	Heat	$0.012
7.	- medium density	0.926-0.940	2,000-3,500	50-300			50-650	0.01	E	F-G	220	- 70	Heat	$0.019
8.	- high density	0.941-0.965	2,400-6,100	15-300			10-650	Nil	E	F-G	250	- 70	Heat	$0.018
9.	Polyethylene-Vinyl-Acetate	0.94	3,100	280	420 graves		600	Nil	E	E	2-100	-100	Heat	$0.02
10.	Polypropylene	0.885-0.900	4,500-10,000	32-1750			300-500	0.005	E	F-E	190-220	- 60	Heat	$0.036 [12]
11.	Polyurethane	1.24-1.26	7,000-9,000		350-600		550-650	0.55-0.77	G	F-E	190	-100	Solvent Heat	$0.13
12.	Polyvinyl Chloride	1.20-1.45	1,400-6,400	60-1400	110-490	20	150-500	Nil	E	G	150-200	- 50	Adhesive Heat	$0.03-$0.05
13.	Polyvinyl Fluoride	1.38	7,000-19,000	12-40		19-70	110-260	0.5	E	E	220-250	-100	Adhesive Heat	$0.29 [12]
14.	Vinylidene Chloride - Vinyl Chloride Copolymer	1.20-1.68	8,000-20,000	10-7100	80-465	25-35	20-140	Nil	E	G	200-300	0	Adhesive Heat	$0.095
15.	SL-64-22-1	0.9	1,800-2,600				200-270	Nil	E	G			Heat	$0.03

Key: P = Poor; M = Medium; F = Fair; G = Good; E = Excellent

1/ See notes on following page.

Table IV-3

PROPERTIES OF SYNTHETIC FILMS

Sheet 2 - Notes

1/ The values reported in this chart are mainly for films up to 10 mills in thickness. Special grades of these materials may be obtained which excel in some particular property.

2/ ASTM D882, procedure B

3/ Elmendorf test. Figures represent pull required to continue tear of 1 mil thick film after starting. Force required to start tear is too high to measure on same equipment and therefore is not normally determined.

4/ SPI-ASTM D100A, static-weighing method.

5/ ASTM D774

6/ ASTM D576

7/ Some of these compositions are the thermoplastic materials that gradually become softer as the temperature increases. The maximum service temperature will depend on formula of material, design and service conditions such as amount of stressing, humidity, etc. Where a range of values is given in the chart, the low value is the limiting temperature for continuous exposure and the high value for intermittent exposure.

8/ The parenthesis indicate that the value is an estimate and not an actual test result.

9/ For 1/32 inch thickness.

10/ For a 1/8 inch thickness.

11/ gm/(100 m^2) (hr) at 103°F., 95% RH

12/ Predicted cost - not now manufactured in thicknesses greater than 2 mils.

or multifilament. Monofilament yarns are composed of solid single strands, and multifilament yarns are composed of several separate fibers spun together. Multifilament yarns are apt to be more flexible, but are also apt to have higher water absorption tendencies and less resistance to ultraviolet light deterioration than monofilament yarns. The most common plastics used for synthetic fibers are nylon (6 or 6/6), polyesters, and polyacrylics, although other such plastics as vinyl polymers, vinylidene polymers, polyethylene, polyprepylene, and polyfluorocarbons are also used. In addition to plastic fibers, there are numerous natural fibers such as cotton, jute, wool, silk, etc.; semi-synthetic fibers such as rayon and cellulose acetate; and inorganic fibers such as fiberglass, asbestos, and metallic strands which are available for weaving fabrics. Natural fibers such as burlap and cotton have long been used in the manufacture of sand bags. However, since they are very susceptible to water rot, microbe attack, and destruction by animals, they are considered unsuitable for such applications as riverbank protection and will not be discussed.

As mentioned in the first two sections of this chapter, most of the previous applications of synthetic materials for erosion protection have been in the form of fabrics. Much of this work has been done in Holland and Germany. The materials tested have included various nylon formulations, polyesters, polyacrylics, polyvinyl chloride, polyethylene glass fibers, and linen (Ref. IV-31). In this country monofilament yarns made out of polypropylenes and polyvinylidene chlorides have also been used in similar applications. The more important fibers which could be used in weaving fabrics suitable for riverbank protection and which are discussed in this section are as follows:

Nylons	Vinylidene Polymers
Acrylics	Polypropylenes
Polyesters	Glass
Vinyl Polymers	

In general, fabrics offer three advantages over sheeting in the proposed applications of synthetic materials in riverbank stabilization. Their permeability can be controlled or pre-determined; they are apt to have greater flexibility and pliability; and they are apt to be stronger. However, because an extra operation is required to weave the extruded material into a fabric, they will be higher in cost.

All of the synthetic fibers described in this section are made from thermoplastic materials and therefore can be sealed and jointed with heat. However, sealing and jointing by sewing may be more efficient and will give a stronger seal. Either might be utilized in joining these materials in riverbank protection.

Nylons (Polyamides): Nylon fabrics (especially Nylon 6/6) have been found by the Europeans to give the best results in such erosion control applications as sand bags and mattresses because of their very high strength and abrasion resistance. Other advantages listed (Ref. IV-19) are high folding endurance, softness, pliability, high elasticity, alkali resistance, and rot resistance. Typical fabrics supplied in this country have a tensile strength of 6.8 to 7.5 of gms/denier. They can be expected to lose from 10 percent to 13 percent of this when submerged. It has a good elongation before rupture (19 percent to 30 percent) but as indicated elsewhere can suffer seriously from ultraviolet deterioration. The approximate cost when woven into fabric having a dry strength of 370 pounds per inch is $0.09 per square foot.

Acrylics: Polyacrylic fibers (e.g., DuPont's "Orlon," Chemstrand's "Acrilan," American Cynamid's "Creslan," and Union Carbide's "Dynel") are superior to nylon in ultraviolet, weathering, and rot resistance. They are not as strong as nylon, averaging 5.2 gms/denier (Ref. IV-24), but they are apt to maintain more strength when submerged (from 90 percent to 95 percent). They have an elongation of 25 percent to 35 percent before breaking. Their cost (per pound of material) is a little less than that for nylon. Chemstrand is currently developing an acrylic fiber, specifically for use in hydraulic applications, which they believe will have the necessary durability and strength for use as riverbank protection. They estimate that the cost of a woven fabric having a dry strength of 250 pounds per inch would be about $0.11 to $0.16 per square foot.

Polyesters: Polyester fibers (e.g., DuPont's "Dacron," Celanese's "Fortel," Eastman's "Kodel," and Beaumit Mill's "Vycron") have tensile strength (averaging about 4.2 gms/denier; Ref. IV-24); which are lower than those for both nylons and polyacrylics. They are very resistant to water, maintaining 100 percent of their dry strength when submerged. They are slightly more resistant to sunlight deterioration than nylon (less so than polyacrylics) and have good abrasion and rot resistance. Elongation at rupture varies from 19 percent to 26 percent. The woven material has high form stability and tear resistance.

Vinyl Polymers: Polyvinyl Chloride and Polyvinyl Chloride-Vinyl Acetate Copolymer fibers have lower strengths than any of the three fibers previously discussed, (averaging 4.0 gms/denier; Ref. IV-24), but they have superior resistance to sunlight and weathering deterioration. They are not affected by water, retaining 100 percent of their dry strength when submerged. Their elongation at rupture varies from 18 percent to 25 percent. These fibers are often used as monofilament yarns. The cost of fabrics woven from monofilament vinyl yarns similar to the "Filter-X" and "Poly-Filter X" described below is estimated to be approximately $0.09 per square foot.

Vinylidene Chlorides: Polyvinylidene Chloride Copolymer fibers have somewhat lower strength than any of those previously discussed (averaging 2.0 gms/denier; Ref. IV-31), but they have fair abrasion resistance and retain 100 percent of their strength under water. They have an elongation of from 27 percent to 30 percent before breaking. Fabrics woven from monofilament yarns are used extensively in such applications as automobile seat covers; and Carthage Mills, Inc. of Cincinnati, Ohio, weaves such a fabric (called "Filter-X") for use as a filter below slope and beach protection. This material has a Mullens burst strength of 210 to 275 points per square inch; a tensile strength of 200 pounds per inch with the warp and 103 pounds per inch with the filling; a tear strength of 80 pounds with the warp and 25 pounds with the filling; and costs $0.09 per square foot.

Polypropylenes: Polypropylene fibers are somewhat stronger than polyvinylidene chloride fibers. Their higher strength, relatively lower weight and water, rot, and mildew resistance have led them to be used extensively (in monofilament form) in such applications as marine rope and netting. They retain 100 percent of their strength when submerged and have an elongation of approximately 25 percent before breaking. Carthage Mills also manufactures a filter cloth ("Poly-Filter-X") out of monofilament polypropylene yarn which is said to be especially abrasive resistant. It has a tensile strength of 335 pounds per inch with the warp and 215 pounds per inch with the filling; a tear strength of 335 pounds per inch against the warp and 95 pounds per inch against the filling; a puncture resistance that was beyond the range of the testing machine (400 points) and costs $0.10 per square foot. They have fabrics woven from multifilament yarn which have a tensile strength greater than 400 pounds per inch with the warp and the filling; a bursting strength greater than 400 Mullen's points; and a cost of from $0.11 to $0.16 per square foot.

Glass Fibers: Glass, in fiber form, has a very high strength, but it is brittle and loses strength rapidly when submerged. Tests were conducted at the University of Florida to evaluate the feasibility of using woven fiber glass fabric or fiber glass matting as a filter beneath riprap for slope or coastal protection (Ref. IV-13). It was found that the material was too brittle and tended to disintegrate rapidly under the conditions experienced. Thus it was concluded that their use in such applications was not feasible. However, if the glass fibers are covered with some other impermeable protection, they retain much of their strength and are probably sufficiently flexible to allow them to be used in the proposed methods of riverbank stabilization. The Chicopee Manufacturing Company of Buford, Georgia, manufactures fiber glass strands where the fiber glass is covered with a polyethylene or vinyl coating. These are used in weaving normal house screening. Strands of 12 mil thickness (and having a strength of nine pounds each) are used in weaving mesh sizes down to 24 by 24 (having a strength of 216 pounds per inch). Strands of 10 mil thickness (having a strength of five pounds each) are used in weaving mesh sizes of 30 by 30 (having a strength of 150 pounds per inch). The manufacturer has woven test samples on a tight fabric (36 by 36 and 30 by 24) out of 12 mil fiber glass strands which have a strength of from 214 to 280 pounds per inch with the filling and 126 to 223 pounds per inch with the warp. The Mullen burst strength ranged from 290 to 402 pounds per square inch. Such a material would cost approximately \$0.10 per square foot. The 24 by 24 weave material above costs \$0.094 per square foot.

Others: Practically any thermoplastic material can be produced in multifilament or monofilament yarns, and such materials as polyethylene and polyfluorocarbon fabrics are already available and have special applications. However, the properties of these fabrics will be very similar to the properties of the films produced from the same materials, and their costs will be approximately in the same relation to one another as those presented in Table IV-3. If fabrics are preferred over films in any proposed application of synthetic materials to riverbank stabilization, cost estimates for the required strength and mesh sizes can be obtained from one of the textile firms engaged in weaving synthetic fabrics, such as Carthage Mills, J. P. Stevens, or Burlington Mills.

Proposed Application and Placement Methods

Some ideas for methods using synthetic sheeting or fabric in lower bank revetment type protection which may serve as a starting point for consideration after the materials to be used have been tested thoroughly have been considered.

Either sheeting or fabrics could probably be used in any of the proposed methods. The sheeting would have to be perforated (except perhaps in Roll Placement Method - II where the anchors could also serve as drains). The size of the tubes in such a mattress will depend upon two factors, (1) the method of filling, and (2) the desired submerged weight. The submerged weight of the mattresses used on the Magdalena River in Colombia has been estimated to be between 25 and 30 pounds per square foot. The larger the tubes, the higher will be the submerged weight of the mattress. However, the larger tubes will also result in a greater reduction in the width of the mattress after filling and thus a higher materials cost per square foot of protected area.

The moisture content of the sand ballast will affect the efficiency of the placement operation. It may be more efficient to fill the tubes with dry sand than moist or saturated sand. The desired moisture condition of the sand ballast will depend upon the method used to fill the tubes, the length and diameter of individual tubes, and the overall economics of obtaining the sand and placing the mattress. Clay would not be a good ballast material, for it would make the mattress relatively impermeable and would be difficult to put into the tubes.

Although dredging at the thalweg of the concave bank is shown as a source of sand ballast in the figures accompanying the placement methods, it may not be the most economical source of ballast. The point bars at the convex bends, the channel at the crossings, and the upper portion of the concave banks are other sources of ballast. If it proves to be a more economical operation to dry the sand before filling the tubes, then the drying apparatus could be placed on a barge which would be required to transport the sand from one of these locations.

One method that was considered and eliminated was the possibility of filling the mattresses and rolling them up on a semi-articulated roller before the mattress is placed on the bank. This method was considered impractical because of the heaviness and bulk of such rolls. If a mattress were to have an average submerged weight of approximately 25 pounds per square foot, to be 350 feet long and 100 feet wide, then it was computed the roll would be about 18 feet in diameter and would weigh approximately 970 tons. Normal construction equipment does not have the capacity to handle such rolls.

Roll Placement Method - I: In this method of placement, illustrated in Figure IV-5, the whole mattress would be prefabricated and rolled up empty on a semi-articulated roller. The top of the mattress would be anchored to the upper bank, and a tug moving away from the bank would unroll the mattress down the slope. In order to prevent the mattress from being picked up and folded over by the current before the sand ballast is placed in the tubes, one or two tubes on the upstream edge would probably have to be filled with a suspension slurry (perhaps plastic soil cement) as the mattress is being unrolled. After the mattress is unrolled, the roller could be either retrieved or left attached to the mattress toe as an additional anchor. The remaining tubes in the mattress would then be filled with a sand slurry from the top of the bank.

The major unknown in this method at this time is that there is no information available concerning the maximum length of tube which can be filled with a sand slurry. In the previous application of Enkalon mattresses (page IV-21) the tubes were fabricated in 16.5 foot sections for filling. The reason for this limitation was that the tube sections were filled on the barge before the mattress was launched, and the launching platform was only five meters wide. Apparently, little trouble was experienced in filling these 16.4 foot sections, but no attempts were made to determine whether the tubes could be made longer. The tubes would have to be filled up to lengths of 800 feet for use on the Lower Mississippi riverbank.

The difficulties with this method of placing at this time are:

1. It is not known whether tubes of such length could properly be filled.

2. Larger dredging pumps would be required than for other proposed methods.

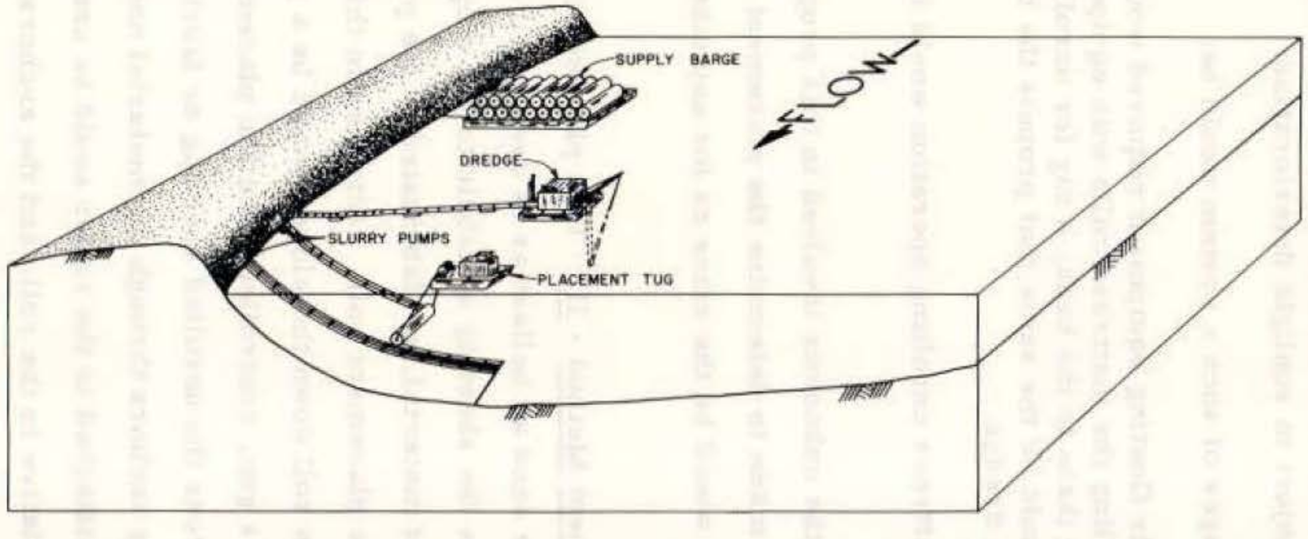

Figure IV-5. ROLL PLACEMENT METHOD-I

3. More time and hand labor would be required than with other methods.

4. Some of the mattress would have to be above water and thus subject to sunlight deterioration.

The advantages of such a system would be:

1. The only floating equipment required would be a barge for hauling the mattress rolls with equipment for transferring them to the bank, a tug for unrolling the mattresses (this could be the same that propels the barge) and a large suction dredge.

2. The mattress unrolling operation would be relatively quick.

Because of the unknowns involved in this proposed method, no attempts have been made to determine the placement cost. The material costs, of course, would be the same as for any other method utilizing such mattresses.

Roll Placement Method - II: This placement method (see Figure IV-6) does not use sand as ballast as is used in all the other methods, but rather anchors the sheeting or fabric to the slope directly. The single thickness of material, prefabricated to the proper size, would be rolled up on the placement roll. Attached to this roll, in such a way that it followed the roll down the slope, would be a guide for the anchor driving gun. This gun, controlled from the placement boat, would move back and forth across the unrolled sheeting or fabric immediately behind the roller, driving anchors through the material and into the bank. A chain mechanism attached to the roller could be used to move the gun at a constant rate relative to the roll, and the anchors could be driven pneumatically or hydraulically. As soon as the top of the anchor was flush with the surface the point could be spread apart to lock the anchor

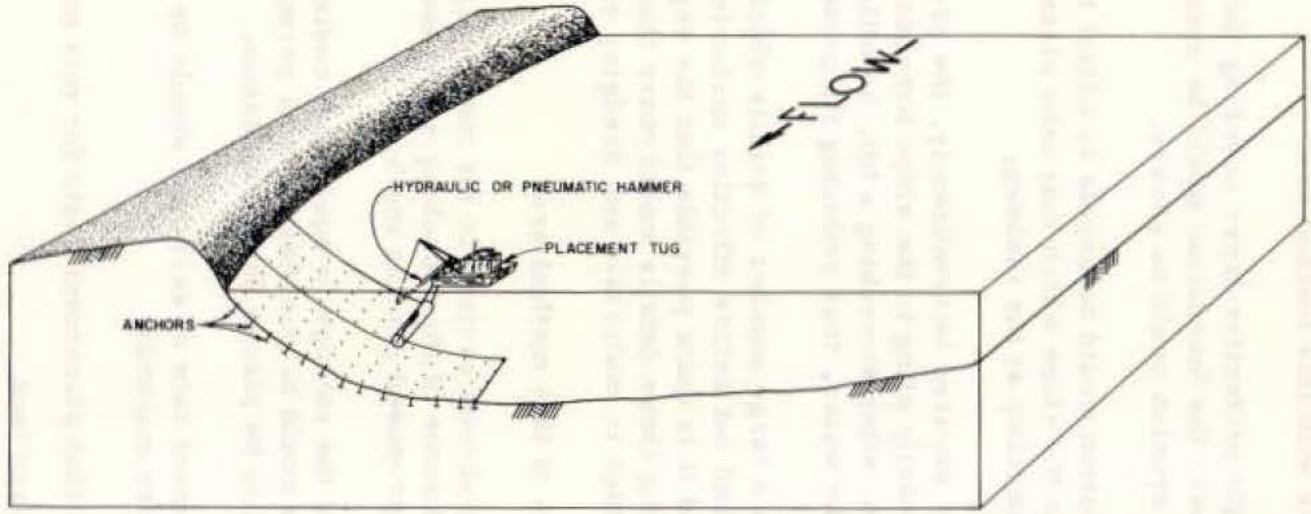

Figure IV-6. ROLL PLACEMENT METHOD-II

into the sand either mechanically or by automatically detonating a small explosive located in the tip.

The difficulties with this method are:

1. If the single protective layer covering the slope were to be ripped, the foundation would be uncovered and the resulting erosion could be severe.

2. The protection would be unable to adjust to severe changes in the slope which may take place after placement, especially at the thalweg.

3. Being only secured intermittently, the protection would not necessarily cling to the slope between anchors and could even, after stretching a bit, be billowed up by the force of the water, thus reducing its protective ability.

4. There are a large number of details which would have to be worked out before effective anchoring could be insured, and it is quite possible that the expenses involved in rectifying these details would more than offset the obvious savings in materials and dredging equipment.

The advantages of this method are:

1. There are obvous savings in the material, dredging equipment, and some of the launching equipment required for the other proposed placing methods.

2. Because of the savings in equipment costs, more placement units could be purchased, thus permitting more protection to be placed during a season.

3. The placement rate of each unit should be higher than that for the other methods.

It is estimated that placement costs for this method could be as low as \$0.05 per square foot.

Barge Placement Method I: This placement method (Figure IV-7) is similar to Roll Placement II, except that the prefabricated mattresses

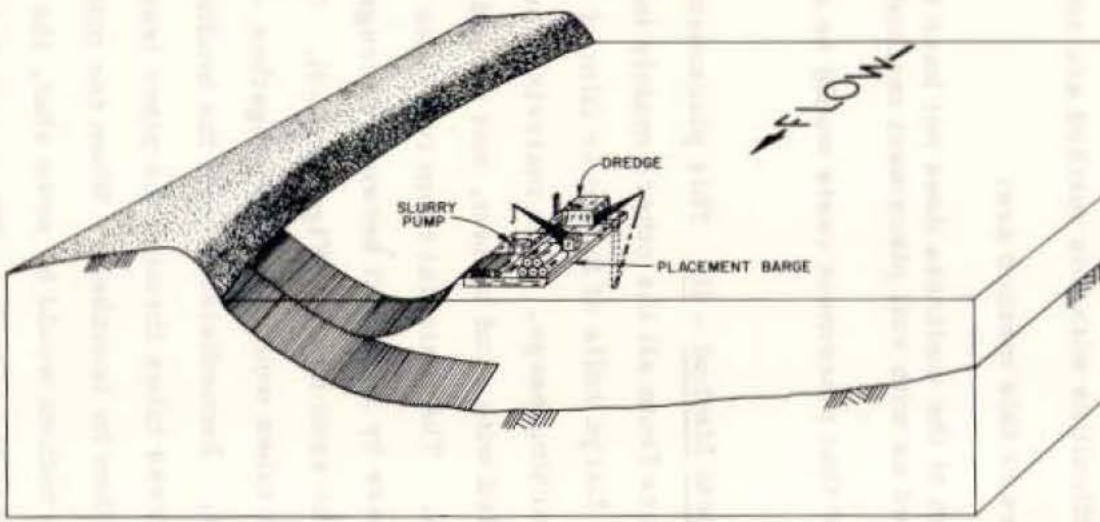

Figure IV-7. BARGE PLACEMENT METHOD-I

are filled right on the launching barge while they are being placed instead of being filled elsewhere and hauled to the bank. The tubes could be filled by blowing sand into them with an air gun such as a press weld gun. Proper filling of the tube could be assured by inserting a mandrel into the tube and extracting it as the tube is filling. Cables would be used to support the mattress during launching.

The major difficulties with this method are not now apparent.

The advantages of this method are:

1. The length of the mattress does not have to be predetermined as with roll placement methods.

It is estimated that placement costs would be as low as $0.08 per square foot.

Barge Placement Method - II: This placement method (illustrated in Figure IV-8) differs from all the others mainly in that the mattress is not prefabricated. Large rolls of fabric or film of single thickness would be stored on the launching barge. The material would then be fabricated into mattresses, filled with sand ballast, and placed on the slope in one continuous operation. The material from two rolls would be formed into the shape of a mattress by pulling it between corrugated plates and the pipes leading from the sand hopper (Figure IV-9). Then two layers of material between the tubes would be bound together -- by sewing fabrics or heat sealing films. Immediately below the binding operation, sand would enter the mattress tubes through the pipes leading from the hopper. The mattress would then be launched. When the mattress had reached a sufficient length, the bottom would be sewn shut, the material cut, and the mattress dropped to the bottom. The top of the next mattress would then be sewn shut, the barge would return to the bank, and the process

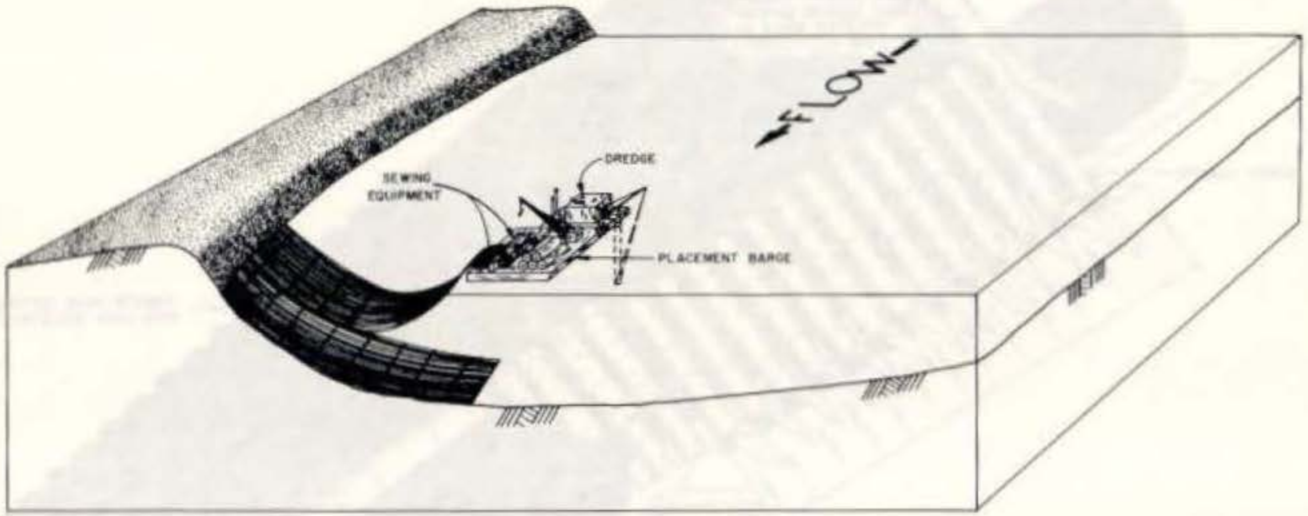

Figure IV-8. BARGE PLACEMENT METHOD-II

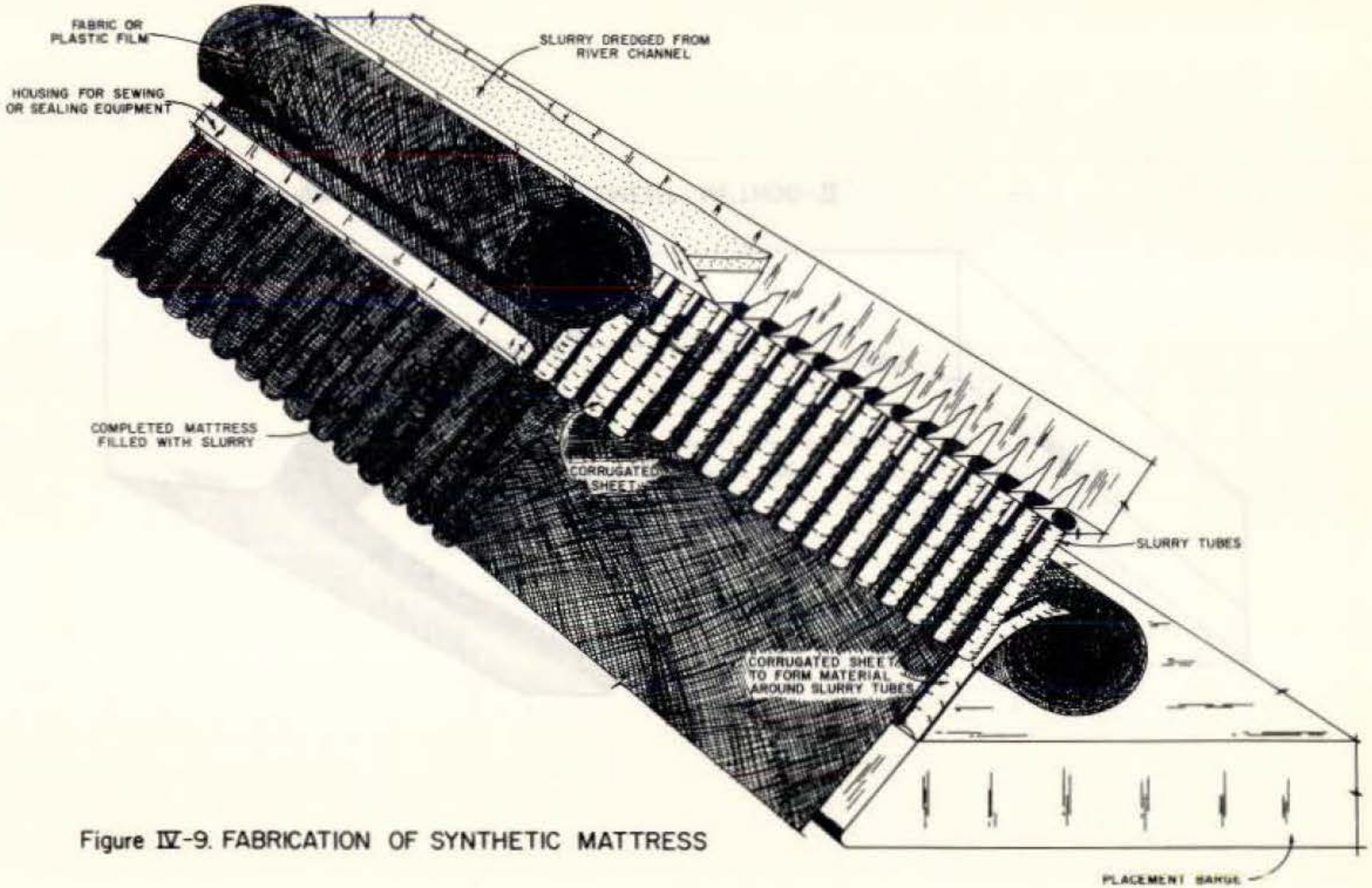

Figure IV-9. FABRICATION OF SYNTHETIC MATTRESS would continue as before, continuously fabricating, filling, and placing.

The sand ballast would be dredged from the river bottom and deposited into the hopper at one end. The excess water would run off the opposite end, and the sand would be distributed through the filling pipes into the mattresses by screws, vibration, or air.

The advantages of such a system would be:

1. The entire operation of fabricating, filling, and placing the mattress would be continuous and almost completely automatic, with a resultant reduction in labor costs.

2. Complete filling of the tubes would be ensured.

3. The size of the mattresses need not be predetermined as with the other systems.

It is estimated that placement costs would be $0.13 per square foot.

PART IV

Chapter 3

MISCELLANEOUS TYPES OF REVETMENTS

General

Some major classifications of materials which have not been studied are those which include natural and semi-synthetic materials. The exclusion of natural and semi-synthetic fibers and sheeting has already been mentioned in Chapter 2. In addition, mattresses woven out of vegetable materials (such as the woven willow and woven lumber mattresses used extensively on rivers throughout the world) have not been considered for the following reasons: (1) information about the manufacturing, physical properties, and cost of these mattresses is readily available, and any further study would add little of significance, (2) these materials have experienced fairly rapid deterioration when used in such applications and therefore are not particularly satisfactory in terms of their durability.

The remaining materials which have been considered for use in lower bank revetments include asphalt, concrete, and metals. In addition, the feasibility of stabilizing the bank materials in-situ and covering the banks with some type of riprap or gabions have been considered.

Asphalt

Asphalt has been used in various applications in protecting the lower banks of the Lower Mississippi and other rivers against erosion. Many of the applications on other rivers have required construction in in the dry and thus would not be feasible on the Lower Mississippi. However, one application method which might be adopted for use on the Lower Mississippi is discussed on page IV-68.

Asphalt has been used on the lower banks of the Lower Mississippi River in three ways. The first two, dumping of mass asphalt and asphalt blocks, have been used mostly in emergency repair work. The third, the asphalt mattresses, was used until 1942 in place of the articulated concrete mattresses in some areas, especially in the New Orleans District (Ref. IV-33, 53). It is reported that the dumping of mass asphalt or asphalt blocks is not being used for permanent lower bank protection because of insufficient strength, durability and erosion resistance; and that the asphalt mattresses are no longer used because of their light weight and relative impermeability (which resulted in launching and securing difficulties), and slow production rates (which was a result of the long cooling times required).

Some of these problems might be solved by utilizing recent developments in asphalt materials. For instance, a medium to rapid setting cationic asphalt emulsion (see page III-27) might be used in place of the hot mix to eliminate the problem of heating entirely. A mix using such a material could probably be produced which would develop sufficient strength in a shorter time than formerly required with the hot mix. An alternative method would be to use the asphalt-sulfur mixture described on page III-30. A mattress made with this mixture would require less cooling time because the asphalt-sulfur hot mix develops the same stability at a temperature of 200 degrees Fahrenheit that a normal hot mix develops at about 77 degrees Fahrenheit.

By using one of these materials it might become possible to continuously fabricate and launch asphalt mattresses. A diagramatic sketch of such an operation might be set up is shown on Figure IV-10.

The first four operations would take place on a continuous conveyor. The first step would be to deposit a strip of the asphalt mixture of sufficient thickness to form the bottom half of the mattress. The second step would be to compact it. The third step would be to cool it or to let it set. The fourth step would be to place the reinforcing. The fifth step would be to place the mixture for the upper half of the mattress. The sixth step would be to compact that layer. The seventh step would be to cool the top layer and perforate the mattress for drainage. The eighth step would be to launch the mattress.

It is estimated that a four-inch thick mattress could be fabricated in this manner for about \$0.42 per square foot and that its in place cost would be about \$0.58 per square foot. This does not include any ballast to keep the mattress in place.

The problem of impermeability is one that could be solved by making more perforations in the mattress. Since the asphalt does not supply any of the tensile strength of the mattress, increased perforations would not have any effect upon this property. The problem of light weight, however, could not be solved as easily. Pure asphalt is not a heavy material having a specific gravity which varies from 1.1 to 1.4. The weight of the asphalt mattresses mostly results from the weight of the aggregate. The mattresses could be made thicker, but this would increase their cooling time and thus reduce the production rate. Probably a more satisfactory method would be to attach ballast to the top of the mattress. Articulated concrete slabs could serve this purpose and could be secured directly to the mattress reinforcing.

The Dutch have experimented with several methods for placing asphalt mattresses (Ref. III-75). These include launching from a barge

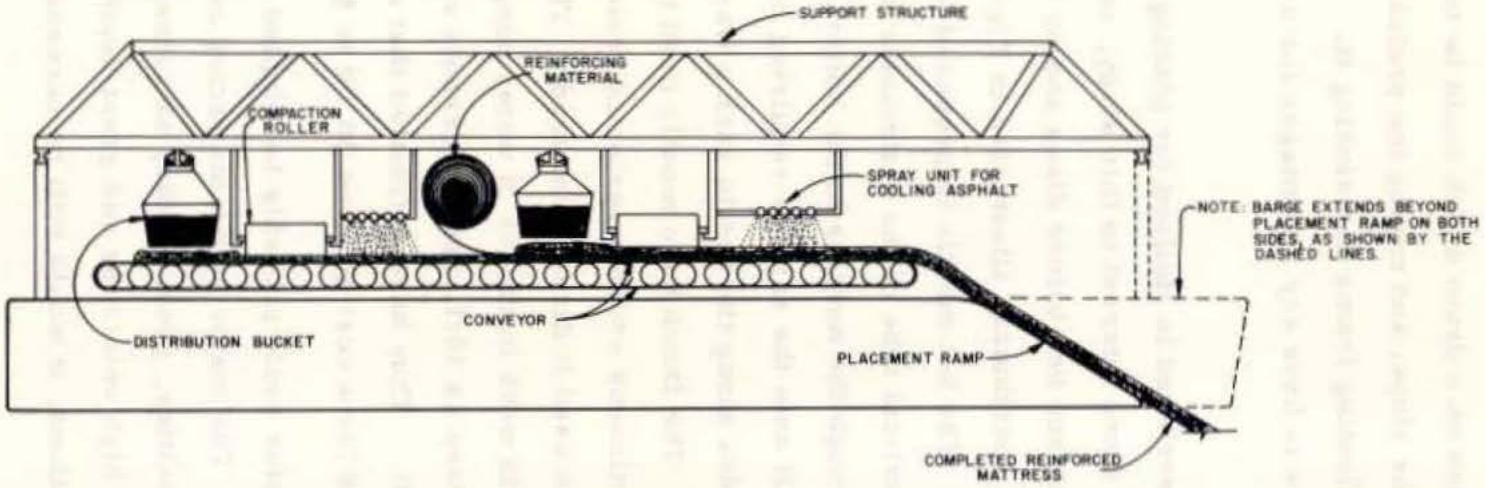

Figure IV-10. FABRICATION AND PLACEMENT OF ASPHALT MATTRESS (similar to the method used by the Corps of Engineers); rolling the prefabricated mattresses on a drum which could be towed to the site, sunk and unrolled down the slope; and towing the prefabricated mattress to the launching site in a floating frame and sinking it. Neither of these latter two methods appears to have any advantages for use on the Mississippi River.

One method developed in Holland for placing asphalt underwater which is perhaps of more interest to this study, is that used to place hot asphalt mastic on the sea bed before dikes and on the bottom of canals (Ref. IV-45). The distributor, illustrated in Figure IV-11, rides off the stern of a ship. The hot mastic is prepared on board the ship and placed in the long vertical tube of the distributor at the same rate as the mastic flows out through the nozzles in the bottom. This maintains a steady flow of asphalt onto the surface requiring protection. The bottom of the distributor rides along the bed in such a way that uniform distribution is maintained. The Dutch have mostly used this method to grout rubble to form a continuous erosion resistant protective covering, although they have also used it directly on sand. The apparatus was designed so that it could work in depths of water ranging from 6 feet to 100 feet, on slopes as steep as 10:1, and in currents of three feet per second or more (Ref. III-38). They have estimated that a two to six inch asphalt layer could be placed for a cost of about $0.09 to $0.20 per square foot.

Such an apparatus could probably be adapted for use on the Lower Mississippi River. The major problems facing such an adaptation include the higher velocities, great depths, and steeper slopes. Assuming that the problems of high velocities and great depth could be solved by mechanical modifications, it would still be necessary to:

(1) Provide drainage facilities.

(2) Develop a material and application method which would provide a covering which would be sufficiently strong and stable to remain on the steep river bank slopes and to resist abrasion and erosion.

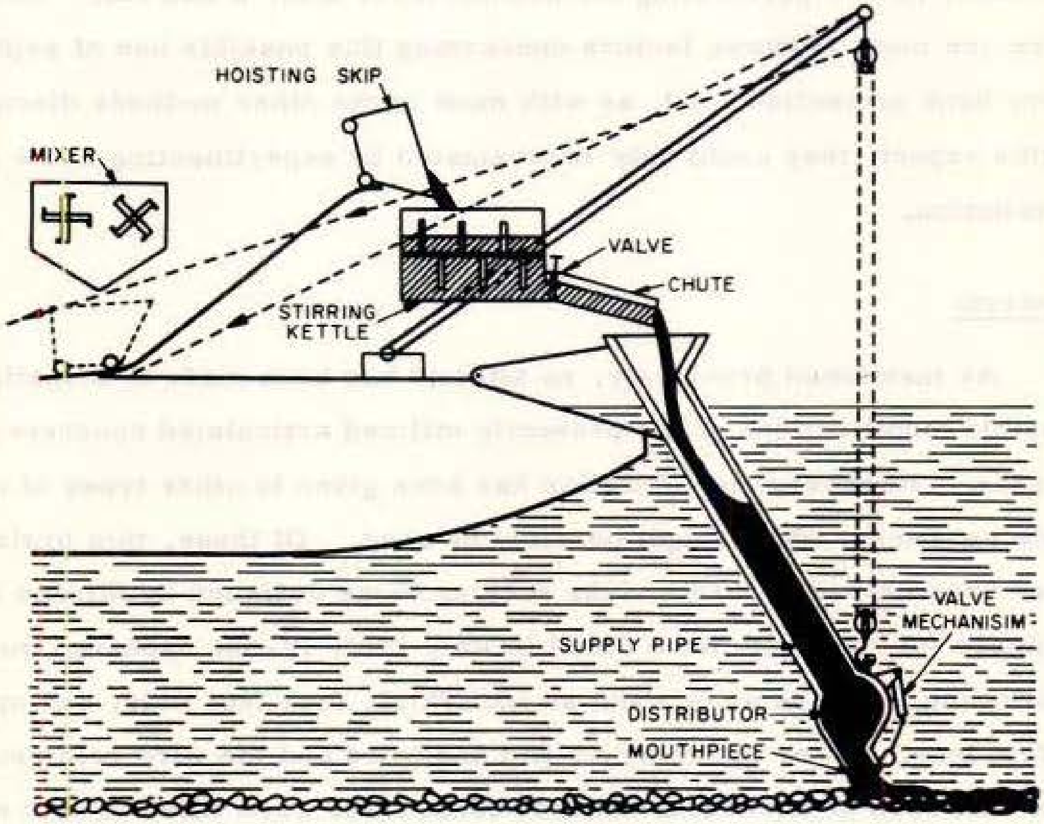

Figure IV-11. MASTIC ASPHALT DISTRIBUTOR

Increased strength might be achieved by placing the mastic on a large mesh, plastic netting reinforcement, such as had been considered for the upper bank asphalt pavement (page III-30 and IV-35). The problem of stability on steep slopes, especially during the placement operation where the mastic is still fluid, might be solved by using either a cationic asphalt emulsion mix or a hot asphalt-sulphur mix as described on pages III-24 to III-30. The problems of permeability might be solved by (1) placing some sort of drainage blanket beneath the asphalt coating which would be open to the river at the toe of the slope, (2) using a mix which was inherently permeable similar to that presently used on the upper bank, or (3) perforating the asphalt layer after it had set. Obviously there are many unknown factors concerning this possible use of asphalt lower bank protection, and, as with most of the other methods discussed in this report, they could only be evaluated by experimenting and a pilot installation.

Concrete

As mentioned previously, no attempt has been made to investigate possible modifications of the presently utilized articulated concrete mattresses. However, consideration has been given to other types of concrete revetment which might possibly be used. Of these, thin prefabricated prestressed concrete slabs such as those designed by Silvano Zorzi as liners for the Ponticorvo Canal in Italy (Ref. IV-38) appeared most interesting. These were about an inch thick, four feet wide, and up to 98 feet long. After the concrete had hardened and the wire reinforcement had been prestressed, the individual slabs were lifted at both ends by a crane and placed on the canal invert which had a parabolic cross-section 66 feet across the top and 21 feet deep. The great flexibility and strength of these slabs indicated that perhaps some similar type of construction could be used as protection on the banks of the Mississippi River.

There are two major difficulties with such construction, however, which discourage such an application. The first is that such thin slabs would probably not have sufficient weight to secure them to the bank, and any thickening of the concrete to increase the weight would result in reduced flexibility. The second is that even though, for concrete construction, the slabs are very flexible, they are probably not sufficiently flexible to allow them to conform closely to normal slope irregularities. This would mean that the entire slope to the depth of the future thalweg would have to be well graded and dressed before they would be used. This does not appear to be feasible with existing equipment.

Cost figures for the prestressed concrete slabs used in the Ponticorvo Canal are not available.

Metals

Consideration has also been given to the possibilities of protecting riverbanks with some sort of metallic sheeting or mattress. The basic costs of metals make such protection obviously economically unfeasible for upper bank protection, but it was decided that consideration should be given to their possible use on the lower banks.

In order for such a covering to be sufficiently flexible to be used as bank protection, it would either have to be in the form of a very thin sheeting, articulated mattresses (similar to the prefabricated landing mats used by the military in expedient airfield construction), or wire fabric.

Aluminum and steel appeared to be the two metals which, economically, might be feasible for use in such protection. Steel, of course, is very susceptible to deterioration by rusting. However, when galvanized and continually submerged in water with relatively little free oxygen, it can last a fairly long time. In river construction, usually a minimum thickness of one-fourth inch is specified for structural members. Costs, of course, preclude the use of such a thick sheeting for revetment purposes. If galvanized, the thickness could be reduced perhaps as low as 0.05 inches corresponding to an 18 gage corrugated metal used in culvert pipe construction. Sheeting made from aluminum alloys is lighter, weaker, and more expensive than steel sheeting. However, it has better corrosive resistance and would probably provide a longer lasting protection.

Metallic sheeting would probably not provide sufficient flexibility unless it were heavily ballasted. Any bending in one axis would increase the stiffness in the other. Because of the cost of the materials, the sheets would have to be relatively thin and only one layer could be used. Costs eliminate the possibility of forming metallic mattresses to be filled with sand ballast.

The light weight of single sheets would create serious difficulties in placing it from a barge because of the high currents in the river. Therefore, it would probably have to be placed in a manner similar to Roll Placement Method II (see page IV-56). The light weight would also create significant difficulties in securing it to the slope. This would have to be accomplished by anchors as in Roll Placement Method II or by adding ballast.

Although such revetments could possibly be constructed and placed, they probably would not offer exceptionally good protection to the banks. In addition to the problems of flexibility and light weight discussed above, sheeting, especially aluminum sheeting, of such thin gage could be easily torn. Thus metallic sheeting protection would be only slightly more effective than a single layer of some of the synthetic films, and would cost much more. Galvanized steel sheeting of 18 gage thickness costs approximately $0.16 per square foot, and aluminum sheeting of the same thickness costs approximately $0.25 per square foot.

The other possible application of metals in lower bank protection was in the form of woven fabrics. Such fabrics would be much more flexible than the steel sheeting and would have higher puncture and tear strength than the synthetic fabrics. They are too expensive to use in fabricating mattresses but could be applied similarly to synthetic fabrics in Roll Placement Method III.

The articulated mattress would be too expensive to be competitive with the present protection and it would not provide sufficient filtering properties. Placement might also be a problem.

Stabilized Soil

The possibility of stabilizing a layer of soil parallel to the existing riverbank but somewhat behind it to form a future revetment has also been considered. The use of chemical and asphalt grouts in stabilizing sands is discussed on pages IV-81 to IV-84 in the following chapter. There are two major problems with such an application of grouting. The first is that grouting a continuous layer on a stable slope of 3h:1v would be very difficult and expensive. The second is that it is not possible with any of the grouting mixtures in common use, to retain permeability in the soil while supplying any significant amount of cohesion between the particles. It is possible that a special cationic asphalt could be designed to do this, but none is known to be presently available. There is a special resin grout developed by California Research Corporation which is reported to be in use in stabilizing the sand walls of oil wells which maintains the soil permeability while increasing its strength. This grout, however, is quite expensive and relies on the higher temperature of the soils at great depths to make it gel. Cost of this grout if produced in bulk might be as low as $1.50 per gallon. Therefore, it does not appear to be either economically or technically feasible to utilize it in such riverbank stabilization, and the same has been concluded for all other available grouts.

Riprap

Riprap has often been used for protecting the lower banks on other alluvial rivers and is occasionally used on the Lower Mississippi. The riprap foundation mattresses used on the Middle Mississippi have already been described in Chapter 2 of Part II, and are reported to be quite successful. In India and Pakistan extensive use is made of a similar construction which is called a fall apron.

The specifications for the riprap to be applied below water on the Lower Mississippi are the same as those for the riprap used on the upper bank (see Part III, page III-1) except that no pieces under six pounds are permitted and the gradation is as follows:

150 pounds to 200 pounds	5% maximum
125 pounds to 149 pounds	5% to 15%
75 pounds to 124 pounds	15% to 40%
25 pounds to 74 pounds	35% to 55%
Under 25 pounds	10% maximum

Concrete and asphalt blocks are also occasionally used as riprap. The concrete blocks are to be approximately 6 inches by 11 inches by 14 inches in size and are to weigh 70 to 80 pounds each. The asphalt blocks are to be approximately 5 inches by 16 inches by 20 inches in size and are to weigh from 85 to 100 pounds each. The thickness of the riprap protection to be placed underwater is specified by the District Engineer.

In most cases such protection will only be placed near the upper bank and is not expected to cover the entire subaqueous slope. Riprap will not be adequate protection at the toe of the slope near the thalweg. As the channel deepens, the riprap will be undercut, forming a gap in the protection.

If it is decided that riprap type lower bank protection could be used more extensively than it is now, some of the artificial riprap materials discussed in Chapter 1 of Part III could be used. The size would have to be somewhat increased to make them consonant with the gradation given above. Sand bags could also be used in such protection. Such sand bags would be very similar to those considered for use in the construction of impermeable groins as discussed in the following chapter.

Gabions

The use of Gabion type units in building groins and other indirect protection construction are discussed in Chapter 4. Similar units could be used in revetment type lower bank protection by laying them next to one another along the slope. Their action would be very similar to the sand filled mattresses discussed in Chapter 2 of this part, and they appear to offer no advantages over this type of unit.

PART IV

Chapter 4

INDIRECT UPPER BANK PROTECTION

General

Indirect bank protection, sometimes called intermittent or non-continuous bank protection, protects the bank primarily by either deflecting from it or slowing near it the erosive currents of the river (see Chapter 2, Part II). This is in contrast to direct or revetment type bank protections discussed in Chapters 1 to 3 of this part which protect the banks primarily by covering them with an erosion resistant layer and do not directly influence the direction or speed of the river current. Such construction as bank heads, impermeable groins, permeable groins, training dikes, and current deflectors are considered indirect protection.

Indirect methods devices are being widely used. Impermeable groins and permeable groins or jetties are used in both this country and abroad. The materials most widely used in their construction are, for impermeable groins, natural stone and, for permeable groins and jetties, timber or steel. Impermeable structures are often constructed with a sand or clay core protected by heavy riprap or other covering. In this chapter other materials or methods which might be used in impermeable construction will be discussed. Materials that might be used in the construction of impermeable units in the bank soils are also discussed.

Impermeable Construction on Existing Foundations

Some of the materials and types of construction which have been considered for such impermeable structures are synthetic riprap, sand bags, gabions, and mattresses.

Riprap: Impermeable structures are often constructed of quarried stone dumped into the river. The stone has to be larger than that used in riprap slope protection because of the higher currents and eddies acting thereon.

Most of the forms of artificial riprap discussed in Chapter 1, Part III and in Chapter 3, Part IV could be used in such construction. The main exception to this is the ceramic riprap. Ceramic riprap cannot be manufactured efficiently in the large units required. However, the small individual units could be fused together as suggested in the 1949 Corps of Engineers study (see page III-7). The fusing process could be accelerated by placing thermite between the ceramic units. Another possibility is tying them together or placing them in a container as described under Gabions.

Another material which might economically be used is concrete. Because of its greater strength, concrete can be made into special shapes such as tetrapods which have superior interlocking tendencies to quarried rock, thus allowing the use of smaller units to resist the same forces. For example, in a sea wall constructed with concrete tetrapods, it was possible to use units weighing 40 percent less than would have been required with stone.

Sand Bags: For reasons mentioned previously (see Chapter 2, Part IV) only synthetic sheeting and fabrics have been considered for use in fabricating sand bags for use in impermeable structures. Some of the synthetic materials which could be used in fabricating such sand bags are discussed in detail in Chapter 2 of Part IV.

Sand bags fabricated from synthetic fabrics (mostly nylon) have been used much more widely in Europe (especially Holland and Germany) than in the United States. The Dutch have used them quite extensively in both temporary and permanent dike construction and in the emergency repair of breached dikes. The factors which they apparently find most attractive are their high strength, resistance to rotting, and resistance to bacterial and animal destruction. Their high strength allows the use of very heavy bags which better resist displacement by high currents and wave forces. The heavy bags also allow a more economical utilization of the capacity of modern construction machinery. Bags weighing one and one-half tons apiece have been used, and experiments with bags weighing up to ten tons are reported. The bags are placed with clam shell buckets with the teeth removed or similar equipment. Their resistance to deterioration allows the bags to be stored for emergency use and allows them to be reused several times.

Since small bags have less tendency to interlock than stone, they may have to weigh more than the stone riprap they might replace. However, there is some adhesion between bags made of synthetic materials that may compensate for the lack of interlocking.

The Germans have used synthetic fabric bags for the same purposes as the Dutch. They have experimented with different types of yarns (Ref. IV-17, 31, 32) but have also concluded that nylon is probably best because of its tear and abrasion resistance. They have generally used smaller bags than the Dutch (about 0.8 tons) because of handling ease and recommend filling them to 78 percent of their maximum theoretical volumes to obtain the optimum combination of weight and flexibility.

Various methods of closing the bags have been experimented with, and it has been concluded, by the Dutch, that a filling nozzle which can be tucked under is most efficient. Allied Chemical Corporation has developed an envelope type opening which appears to have merit.

The size of the bag to be used depends upon a number of factors, among which are:

1. Anticipated force of current (establishes a minimum)
2. Strength of fabric
3. Cost of fabric
4. Strength of seams
5. Cost of sealing bag
6. Capacity of placing apparatus
7. Filler used (bag of damp angular sand will handle easier than those of dry or saturated rounded sand (Ref. IV-31).

In reference IV-31 several tables and graphs are presented which show the relation of bag dimensions versus weight and volume. For use in any particular application, the size of bag to be used will depend upon the unique evaluation of the above factors.

The bags can be filled with any cohesionless material available along the river.

The cost of the bag material will be approximately the same as given in Table IV-3 (page IV-46), with some small additions perhaps added for special weaving or seaming.

Gabions: These are defined as sausage or box shaped containers filled with ballast for use in river channel and bank stabilization. They are often used in the regulation of small rivers in Europe where large stone is not available. The container can be constructed of chicken wire, plastic, or willows, and is often filled with gravel and cobbles to form check dams, sills, or groins. A European firm, Maccaferri, has established offices in this country to market gabions for use in slope protection and channel stabilization. They produce both a plain wire fabric gabion and a plastic coated wire fabric gabion, which is more rust resistant. These units, with their large mesh, could be filled with imported stone or one of the types of artificial riprap discussed in Chapter 1, Part III and Chapter 1, Part IV.

Gabion type units can be constructed from fabric or sheeting. Fabrics can be woven in a continuous tube with a special lock stitch which prevents "running." Thermoplastics can also be extruded in a continuous tube which can be cut off and sealed in short sections to form bags or in longer sections to form gabions.

The advantages of gabions over bags in such construction as an impermeable groin is that each unit has more weight and the various units tend to interlock more. However, gabions would probably be more difficult to place in the great depths found in the Lower Mississippi River.

Mattresses: Mattress or revetment type protection, as discussed in Chapters 1 to 3 of this part, could be used in the surface protection of impermeable construction built out of erosion susceptible materials such as fine sand. An interesting experimental groin construction using mattresses on the Lower Mississippi was briefly mentioned in Reference B-31. According to this report, mattresses about 200 feet wide were placed on the concave bank at intervals. The area between adjacent mattresses was left unprotected. As the unprotected material was eroded away from between the mattresses they were somewhat undermined at their edges. Thus the center had a somewhat superior elevation to the edges and the structure became, in affect, a submerged groin. No further reports on the success of these experimental installations have been obtained.

Impermeable Construction In-Situ

In addition to investigating the possibility of constructing impermeable structures on existing foundations, consideration was also given to the feasibility of stabilizing soil masses in-situ to increase their strength and erosion resistance. As these masses are uncovered by the erosion of the covering soils, they would have the same effect as the impermeable structures. Two methods of construction have been considered. The first is to use some type of grout to stabilize the soil, and the second is to remove this soil and replace it with concrete or to mix the soil in place with cement. These two construction methods are discussed in the following paragraphs.

Grouting: Generally, cement grouts are not capable of effectively penetrating and stabilizing deposits as fine as those found in the banks of the Lower Mississippi River. Therefore, it would be necessary to use a less viscous grout. The most common type grout normally used under such circumstances is a combination of chemicals which are naturally fluid or have been dissolved in water having a very low viscosity after mixing and during injection; and which after a predetermined time, react to form a gel. Many of these grouting compounds (such as AM-9) are used primarily to eliminate the seepage of ground water through the treated deposits. They form a gel which, though impermeable, adds very little strength to the soil. Others, such as the various sodium silicate mixtures and the urea-formaldehyde grouts, add significant strength to the soil in addition to making it impermeable. The urea-formaldehyde grout (e.g., Halliburton's Herculox and American Cyanamid's Cynalox) are quite recent developments and, although they seem to be about the strongest of the commonly available grouts, are reported to be difficult to use effectively and to have uncertain durability. Their cost discourages their use in riverbank stabilization. The oldest sodium silicate grouting mixture, called the Joosten process, provides high strength but is difficult to use. In this process the two chemicals, sodium silicate and calcium chloride, react almost instantaneously upon mixing and therefore must be injected separately. This two-shot method increases the grouting costs. The speed of the reaction limits the zone of effectiveness to a small distance from each injection point. A modification of this method is one in which a less reactive agent replaces the calcium chloride allowing the two chemicals, with the addition of a retarding agent, to be mixed together before injection. Such a grout is manufactured by the Diamond Alkali Company under the trade name of "Siroc." The gel formed by Siroc has less strength than the gel from the Joosten process or urea-formaldehyde grout. A mixture in which the sodium silicate is diluted to 50 percent of its pure strength with water is predicted to be strong enough to resist erosion by river currents. The interval between the injection of the mixture and the time it gels can be extended or reduced by varying the amount of retarding agent added. This would allow a longer volume of soil to be stabilized from each injection point. The cost of this grout, with 50 percent dilution is approximately \$0.17 to \$0.21 per gallon.

Another type of grout which might be effective but which is still in a developmental stage is one using cationic asphalt emulsion (see pages III-25 to III-30). As has been explained previously, the asphalt globules in such an emulsion can be made so small that they will permeate medium and fine sand, and their breaking rate can be made dependent upon time, chemicals reaction, or contact with aggregate. The development of a cationic asphalt emulsion grout for use in stabilizing the sands around oil wells which is almost completely dependent upon contact with aggregate has been reported. This grout is said to continue to permeate into a sand stratum until all the asphalt globules have been depleted by attaching themselves to the sand grains. No cost figures are available for this material, but normal cationic asphalt emulsions cost approximately \$0.16 to \$0.17 per gallon. If it is assumed that there is a void ratio of 0.54, that the chemical grouts will entirely fill these voids, and that the asphalt in a cationic asphalt emulsion grout would fill approximately 50 percent of this void, the cost of the grouting materials for such stabilization can be estimated as follows:

Chemical Grouts (assume \$0.19 per gallon)

.35 cubic feet = 2.6 gallon

$2.6 \times \$0.19 = \$0.50$ per cubic foot

Cationic Asphalt Emulsion Grout (assume \$0.17 per gallon)

$$\frac{2.6}{0.65} \times .50 \times \$0.20 = \$0.34 \text{ per cubic foot}$$

There are two major problems with using grout to stabilize in-situ impermeable structures in the Mississippi River deposits. The first is that with existing grouting methods the cost of placing the grout is likely to equal or exceed the cost of the grout itself. To make such an application economical some method of massive grouting would have to be developed. One possible massive grouting method could be adopted from a method patented by Mr. C. E. Peeler of the Diamond Alkali Company (Ref. IV-48). In this method, applicable solely to acid soils, sodium silicate is allowed to flow into the soil through a stand pipe under a small head, where the acid in the soil slowly reacts with it. An adaptation of this method would be to use a grout with an extended gelling time or the special cationic asphalt grout mentioned above. An alternative, which might provide more control over the distribution of the grout, would be to place a number of widely spaced grouting pipes in the area to be stabilized. The grout would be injected under pressure into the structure through alternate pipes, and water would be pumped from the stratum through the intermediate pipes. By pumping the water out of the soil, seepage gradients would be established which would direct the flow of the grout from the input pipes to the pumping wells. Such pumping would be continued until a sufficient amount of grout was being pumped out with the water to indicate that stratum was fairly well filled with grout. A small amount of grout would then be injected through the well pipes to stabilize the immediately surrounding soil; or, alternatively, the operation could be reversed and water pumped out of the former grout pipes until there was a sufficiently low flow to indicate almost complete sealing of the stratum.

No indication has been found during the course of this study that either of the above massive grouting techniques (excepting the limited case covered by the patented) have ever been considered or experimented with. Therefore, their feasibility could only be determined by undertaking large scale testing of both the method and the various grouting compounds.

The second major problem with grouting in-situ structures, however, might make such construction infeasible along the Lower Missis-sipi even if suitable massive grouting techniques using inexpensive grout could be developed. This is the fact that the alluvial deposits are apt to be very variable in permeability. Sand layers could alternate with silt and clay layers. With such a situation it would be practically impossible to completely stabilize the deposits effectively.

<u>Mixing Soil Cement in Place</u>. Consideration has also been given to the possibility of constructing in-situ structures by mixing the bank

sands with cement in place, using a device similar to that patented by the Intrusion Prepakt Company, Inc. for installing mixed in place piles. With this equipment, 15 to 24-inch diameter holes are drilled into the ground with an auger. As the auger is withdrawn, a cement slurry is injected into and mixed with the disturbed soil in the holes, forming a soil cement mixture which hydrates to form high strength and erosion resistance. Existing equipment is mounted on trucks and is capable of drilling holes up to 100 to 120 feet deep vertically or on slight batters (to 1h:3v). Modifications could probably be made to allow other types of mounting and deeper holes to be drilled. The compressive strength of the soil cement can be expected to average approximately 2000 pounds per square inch. The piles can also be reinforced by inserting a wire mesh after the soil cement has been mixed. Adjacent piles can be overlapped to give, in effect, a continuous structure in all three dimensions. The soil cement also need not extend completely to the surface.

It was estimated that groins as shown in Figure IV-12 would cost about $100,000 a piece at the present time. Present in place costs would probably be approximately $4.00 per cubic foot, of which about $0.50 would be for material. It was estimated that after gaining sufficient experience in installing such piles on a large scale under the conditions found on the Lower Mississippi, the total in place cost might be reduced to as low as $2.50 per cubic foot.

The advantage of using this type of construction as opposed to grouting is that it would be possible to ensure thorough stabilization of the soils.

Thermal Stabilization: As previously mentioned (page III-16), silt and clay deposits have been thermally stabilized to increase their

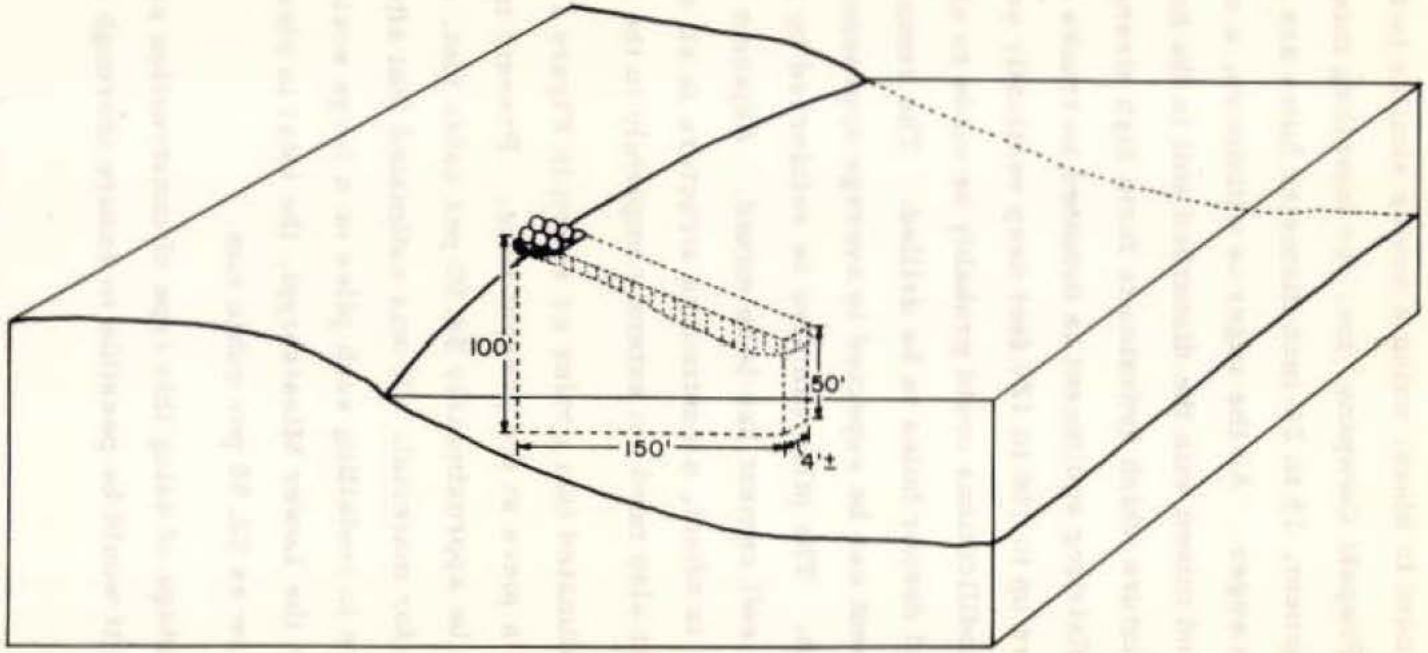

Figure IV-12. TYPICAL GROIN.

strength and bearing capacity by Eastern European engineers. Fuel is burned in holes to stabilize the surrounding soil. Such stabilization could not be accomplished with existing methods in sand, but where a bank was formed exclusively of silt and clays, it might be effective in sufficiently increasing the erosion resistance of the materials to make them work effectively as impermeable indirect protection structures. Because the Mississippi River banks are not normally so composed, however, this idea has not been investigated in detail.

PART IV

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Appendix IV-A

CERAMIC TESTING

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Appendix IV-A

CERAMIC TESTING

Introduction

Before it was possible to evaluate the economic feasibility of using ceramic material produced from clay and silt deposits along the Mississippi River in riverbank protection, it was necessary to determine whether it was technically possible to produce suitable materials. The preliminary attempts to determine this involved extensive correspondence with the geology departments of the states bordering the river and a comprehensive literature search in an effort to obtain information on all previous ceramic testing of these deposits. The result of this correspondence and search, presented in the following sections, indicated that very little such information was available. The information available indicated that these deposits were of doubtful value for normal ceramic processing.

It was then decided that further testing should be conducted under this contract to confirm these data and to determine whether it might be possible to produce suitable ceramics by an alternative method. Soil samples from six typical deposits along the river were obtained through the personnel of the Waterways Experiment Station and shipped by them to Dr. Mueller of the University of Washington for testing. These samples were obtained from the following types of deposit:

Backswamp	- 2
Clay Plug	- 2
Silt Overburden	- 1
Vicksburg Loess	- 1

Table IV-A-1 gives the properties and locations of these soil samples. Figure IV-A-1 to IV-A-6 are gradation curves for the six samples.

Dr. Mueller's report on the two phases of the ceramic testing follows the summary of geological investigations. Following this report are some photographs of the testing.

Geological Investigations

The following is an outline of the pertinent information obtained during preliminary attempts to determine the suitability of Mississippi River silt and clay deposits for ceramic processing.

Kentucky:

A. Kentucky Geological Survey - letter

Have no information concerning Mississippi River soils but experience with other alluvial deposits indicates that they are red burning and could be used for common brick at cone 1.

Tennessee:

A. Reference I-26

Conducted some tests on clay from the Jackson Formation which is not of recent alluvial deposition but outcrops along the riverbank. The deposits are limited and of poor quality. The two samples tested produced fair bricks, the first at cone 9 and the second at cone 3. The tests were discontinued due to the nature of the deposits and because the results were "not distinctly promising."

Arkansas:

A. Arkansas Geological & Conservation Commission
Helena - West Helena Clay Investigation - letter

Mississippi River alluvium was tested and found unsatisfactory due to bloating. However, other deposits were found near the city and tested successfully.

Mississippi:

A. Mississippi Geological Survey - letter

"... some of the alluvial clays mature at a very low temperature and some of them have a tendency to bloat. In commercial practice... the bloating tendencies of clays can be controlled to some degree."

B. Reference I-18

Took several samples and tested them for ceramic properties:

E 52 - "Blue Swamp Mud." Taken in clay plug near old mouth of Yazoo River west of Vicksburg. Fifty percent shrinkage upon drying. Three hundred percent bloating at cone 02. Sp. Gr. = 0.66. Strong. Makes a good blend.

E 1 - "Byram Silt," E 21 - "Byram Clay." Occurs on the Vicksburg bluffs. Bloating tendencies. Weak. Not available at riverbank, but might make a good blend with "Blue Swamp Mud."

E 144C - "Brown Loam," E 155 - "Weathered Brown Loam." Is actually the loess occurring on Vicksburg bluffs. Burns to steel hardness from cone 4 to 5. High specific gravity. Shrinkage 10-25%. Recommendations are that the blue swamp mud be blended with either the Brown Loam or Byram Clay to produce the best raw material for ceramics.

C. Reference I-28

Several samples taken and tested for ceramic properties. Samples are not from recent river deposits but miocene and pliocene formations which outcrop in the river bluffs. These formations are Hattiesburg and Citronelle. Hattiesburg samples G8a, G36, and G501C were tested as bonding clays. Fired at cone 03. All samples were steel hard but cracked. No bloating. Shrinkage 14%. Specific gravity from 1.85 to 2.15. Citronelle samples 11, 96, 115 all tested for ceramic properties. All steel hard at cone 0.3. Shrinkage up to 10% except at higher temperatures (cone 14) where there was a tendency for bloating. Specific Gravity around 2.0.

Louisiana:

A. Louisiana Geological Survey - letter with test results

Samples LGS 32, 33, 34, 35, 36 all located near recently cutoff ox-bow lake in vicinity of Ferriday (probably recent alluvials). All bloated at temperatures below cone 1. Sample 40 located near river in upland pleistocene terrace. No bloating, hard at cone 06, very hard at cone 11.

The information obtained from various soil conservation departments is of a very general nature and not particularly pertinent. All silts and clays deposited by the river are classified in several related series, the most dominant of which is the "Sharkey Series." A general agricultural chemical composition for this series is given and it is stated that,

"Major clay mineral types associated with the Sharkey clay are the same for all locations. However, variation in minor component appears to be quite common."

One significant engineering datum given is that the Sharkey Series has a very high shrink-swell behavoir. This would indicate that it would be unsuitable for normal ceramic production. The associated soil series vary in shrink-swell behavior from very high to low. This behavior seems to decrease with increasing particle size.

Dr. Mueller's Report on Testing

Work Statement

This is the final report on work accomplished upon the utilization of Lower Mississippi River soil samples (clays) as possible materials for bank stabilization. Reference is made to my letters and appendices of July 31, 1963, and September 9, 1963; to Harza letters of August 9, 1964, and October 25, 1963; and letter and enclosures to the undersigned by W. J. Turnbull, Chief, Soil Division, U. S. Army Engineers Waterways Experiment Station, dated December 11, 1963.

The purpose of this work was to test clay samples furnished to ascertain their characteristics relative to conventional forming and firing and pyroplastic forming. Initially, seven clays were to be tested to determine their shrinkage, fired density, fired absorption and bloating characteristics. Subsequently, it was agreed to subject each of the materials to the laboratory equivalent of pyroplastic forming in order to determine the technical and economic feasibility of using this method. Tests performed on specimens prepared in this manner included compression (crushing) strength, absorption and density.

Materials

A total of six clay samples were furnished by the U. S. Army Engineers Waterways Experiment Station in December, 1963. Information regarding these samples was furnished by Mr. W. J. Turnbull, Chief, Soils Division, with his letter of December 11, 1963. (Figures IV-A-1 to 6 and Table IV-A-1.) Of the samples furnished, five of them were obtained within roughly a twenty-mile radius of Vicksburg. The sixth sample (Clay Plug-Ozark Revetment) was obtained approximately 120 miles upstream.

The description of the overburden and clay plug samples indicate that they would be the most readily available for processing since the backswamp clays and the loess are, at times, some distance from the channel. All clays were tested.

Procedures

A. Conventional Forming and Firing. The clay samples, as received, were tested for moisture content. An 8000 gram (17.7 pound) portion of clay was placed in an 18-inch diameter Simpson laboratory mixer and mixed for five minutes in the as-received condition. Sufficient water was added to obtain optimum workability for extrusion, then mixed for an additional 15 minutes to obtain a uniform sample. The clay was then extruded with a Davis-Brown unit using a 1-inch x 1-inch die. The extruder is equipped for de-airing. A vacuum of 26 to 28 inches was used on all samples. The extruded column was cut into 6-inch long bars which were scribed for shrinkage determination and coded for identification. The bars were air dried for at least 24 hours, then placed in a recirculating air drier at 230°F for 24 hours.

After bars from all six samples were prepared, each set was divided into three groups, and each group fired to one of three different temperatures. Existing standards were utilized for possible further comparison of data. The maturing temperature is reported in terms of the

Orton Standard Pyrometric Cone Series. The firings were to cones 05, 01, and 5. A plot of the time-temperature relationships for each firing is given in Figure IV-A-7.

B. Pyroplastic Forming. Since no equipment was available to prepare samples by this method an attempt was made to duplicate the type of finished product that might be obtained by forming by what is considered a conventional hot-pressing technique. Each of the materials was first heated to approximately 1800°F to duplicate the conditions that would exist if the materials were pre-calcined in a rotary kiln or moving hearth furnace. After cooling, the material was placed in a one-inch diameter cavity drilled in a six-inch cylindrical graphite rod. A two-inch Inconel metal plug was placed in the lower portion of the cavity, and a six-inch Inconel rod was placed on top of the material. The entire mold, or die, was placed in an induction coil connected to a General Electric 20-kilowatt induction generator. The mold was heated by induction to temperatures between 1800 and 2100°F. As the mold reached this temperature, pressure was applied to the Inconel plug. The pressure and temperature were maintained for periods of from one to three minutes in order to produce the desired glassy phase in the sample. When the induction generator was turned-off, the pressure was maintained until the temperature of the mold returned to about 1600°F, at which time the pressure was released and the mold cooled to a proper handling temperature. The samples were then ejected and the ends faced off with a diamond cutoff wheel for subsequent testing. This laboratory test procedure most closely approximates the proposed production method. Variations in temperature and time were purposely made on several samples in order to determine their effects upon the final characteristics of the material. A graph showing typical time-temperature relationships utilized in hot pressing the King's Point overburden is given in Figure IV-A-8. All temperatures were measured by a Leeds & Northrup optical pyrometer sighting on the exterior of the graphite mold. These temperatures, therefore, are not the actual temperature of the material being pressed, but they do give a relative value from sample to sample. The proposed pyroplastic forming will obviously not be so time consuming. The clay will be heated prior to pressing, permitting much more rapid pressing rates.

C. Testing. The testing to determine physical properties of the bars fabricated by conventional means and those fabricated by hot-pressing are as follows:

(1) Water Content - A sample of clay or an extruded bar were weighed to obtain the wet weight (W_w), then dried at 230°F and reweighed to determine the dry weight (W_d). The percent water content was calculated on the dry basis using the following formula:

$$\text{Water Content (\%)} = \frac{W_w - W_d}{W_d} \times 100$$

(2) Total Shrinkage - The extruded bars were scribed with two lines six inches apart (L_w). After firing, the distances between the lines were measured (L_f) and the total shrinkage calculated using the following formula:

$$\text{Total Shrinkage (\%)} = \frac{L_w - L_f}{L_w} \times 100$$

(3) Bulk Density - The procedures outlined in ASTM C20-46 were used and the bulk density calculated using the following formula:

$$\text{Bulk Density (gram/cc)} = \frac{\text{Dry Weight}}{\text{Bulk Volume}}$$

For conversion from metric units (g/cc) to U. S. system (pounds/cubic foot), the metric value should be divided by 1.6×10^{-2}.

(4) Water Absorption - The procedures outlined in ASTM C20-46 were used and the percent water absorption calculated using the following formula:

$$\text{Water Absorption (\%)} = \frac{W_s - W_d}{W_d} \times 100$$

Where W_s = saturated weight

W_d = dry weight

(5) Compressive (Crushing) Strength - The compressive load required for failure of the hot-pressed specimens was obtained using a Baldwin testing machine. A constant load of 20,000 pounds per minute was applied and failure was taken at that point where the first breakup of the specimen was observed. In most cases, total failure occurred, although in several specimens a splinter of material broke off one side, leaving approximately 90 percent of the test specimen intact. In the latter cases, this was considered failure.

Results

a. Conventional Processing. A table of measured properties of the samples formed and fired by conventional methods is given in Table IV-A-2. A brief description of the properties of each sample follows:

(1) Loessial (Openwood Street). This material was very fine-grained, and some difficulty was experienced in extrusion since the clay had very little workability or plasticity. The extruded bars were too weak to be handled immediately after extrusion. Vitrification

began between cone 05 and cone 01. Acceptable properties were obtained from the cone 5 fire. No bloating was experienced, and the clay could be considered satisfactory for conventional structural clay products or possibly low grade refractories if dry-pressed forming was utilized.

(2) Clay Plug (Ozark Revetment). This material had an extremely high moisture content as received, yet extruded fairly well. It is a low firing material which had unsatisfactory properties when fired to cone 05. It bloated at the cone 01 firing and would not be satisfactory for production other than lightweight aggregate.

(3) Clay Plug (Willow Cutoff). This material is similar to the Ozark Revetment sample but probably has a lower maturing temperature. It bloated at cone 01 and could be used only for lightweight aggregate.

(4) Back Swamp (Long Lake). This material was similar to the Ozark Revetment sample in that it bloated severely at cone 01 and had no particularly desirable properties at the lower firing temperature.

(5) Back Swamp (Mound). The low temperature firing of of this material (cone 05) produced a bar which was badly fractured and even at this low temperature showed signs of incipient bloating. From a conventional forming and firing standpoint this material would have little or no use.

(6) Overburden (King's Point). This material had satisfactory properties for the production of structural clay ware at the cone -1 firing. The material had not reached its maturing temperature at the cone 05 firing, and incipient bloating was evidenced at the cone 5 firing. This material might be used for conventional ceramic processing but probably has a narrow firing range.

b. Hot Pressing. A table of processing data and measured properties of samples formed by the laboratory hot press method is given in Table IV-A-3 and IV-A-4. The column marked "High Temperature Range" refers to the significant temperature range believed to be required for the vitrification of the material. The column headed "Time at High Temperature Range" indicates the average time (in minutes) during which the sample was within that range of temperatures. Ten specimens were pressed from each sample of clay. In several instances (Ozark Revetment and Willow Cutoff) variations of temperature and pressure were used to observe differences in physical properties.

All hot-pressed samples had considerably higher densities and lower absorption than samples of similar clays processed conventionally. The bloating effects were minimized by calcining and hot pressing. Considering the abundance and proximity of the overburden material to the river channel, the results obtained from that sample are most promising.

No comparisons of crushing strengths are possible since the conventionally processed specimens were not tested for this property. It can be safely assumed that the increase in strength obtained by hot pressing is at least two or three times that which might be expected to be obtained by conventional processing.

Conclusions

A. Conventional Processing.

(1) The Loessial sample could possibly be used to produce a product satisfactory for rip-rap if dry pressing was used in forming and the material fired to at least cone 5.

(2) The Overburden material has possibilities as a rip-rap product produced by either extrusion or dry pressing and fired near cone 01.

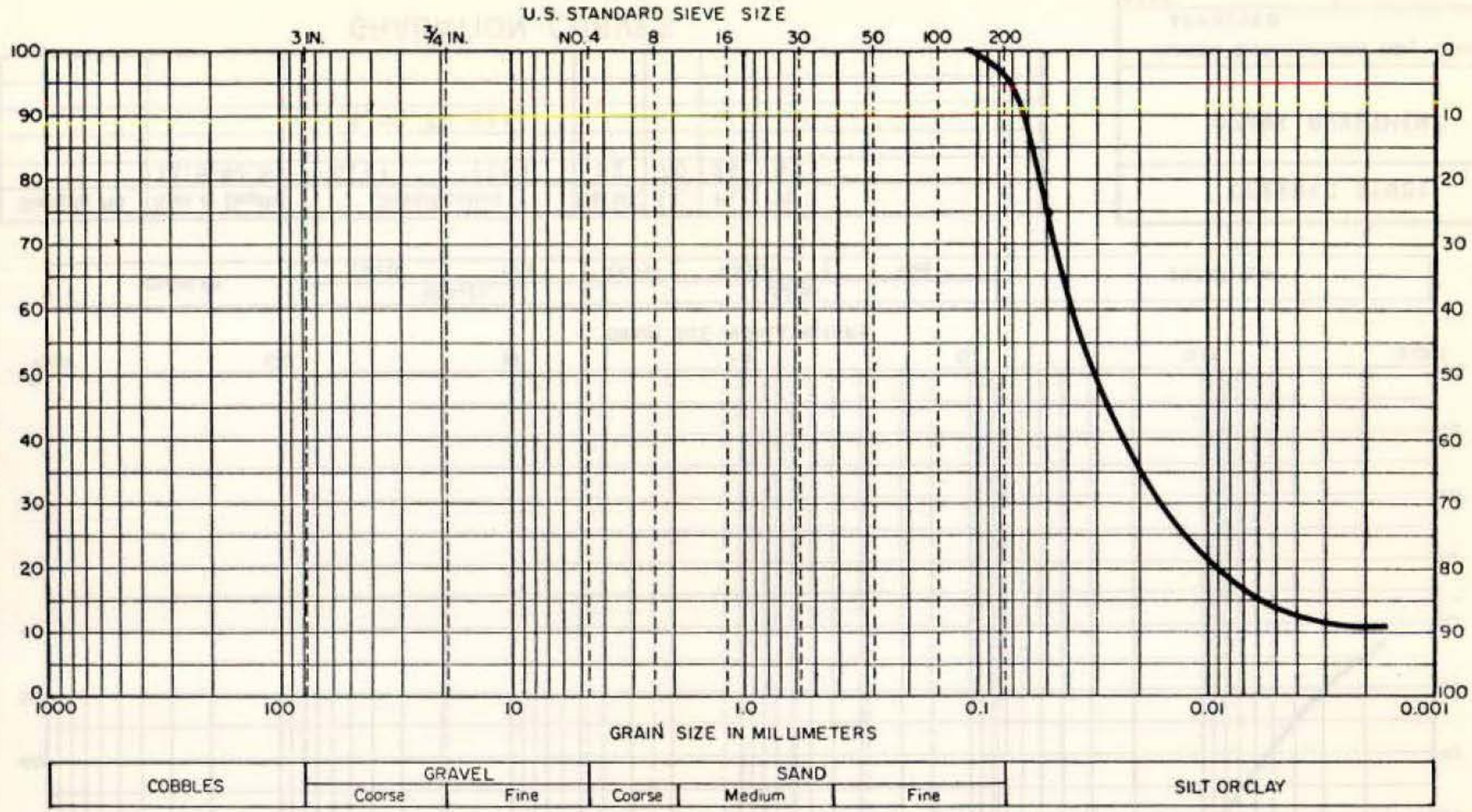
(3) The other samples offer no promise for any product other than lightweight aggregate, which is not desired in this study. It should be noted that the other four samples and the Overburden may have pozzolanic properties if processed properly.

B. Pyroplastic Forming

(1) From the standpoint of availability and properties noted, the Overburden offers excellent potential.

(2) All samples seemed to be satisfactory, although the Loessial sample may be marginal due to its relative refractory nature.

(3) No definite conclusions regarding the use of these materials in pyroplastic forming may be made until further studies are made utilizing the actual equipment in a pilot plant operation.



Sample No.	Elev. or Depth	Classification	Nat WC	LL	PL	PI	
		SILT (ML)	14	28	23	5	
		G_S=2.66 est.					

GRADATION CURVES

CERAMIC STUDY

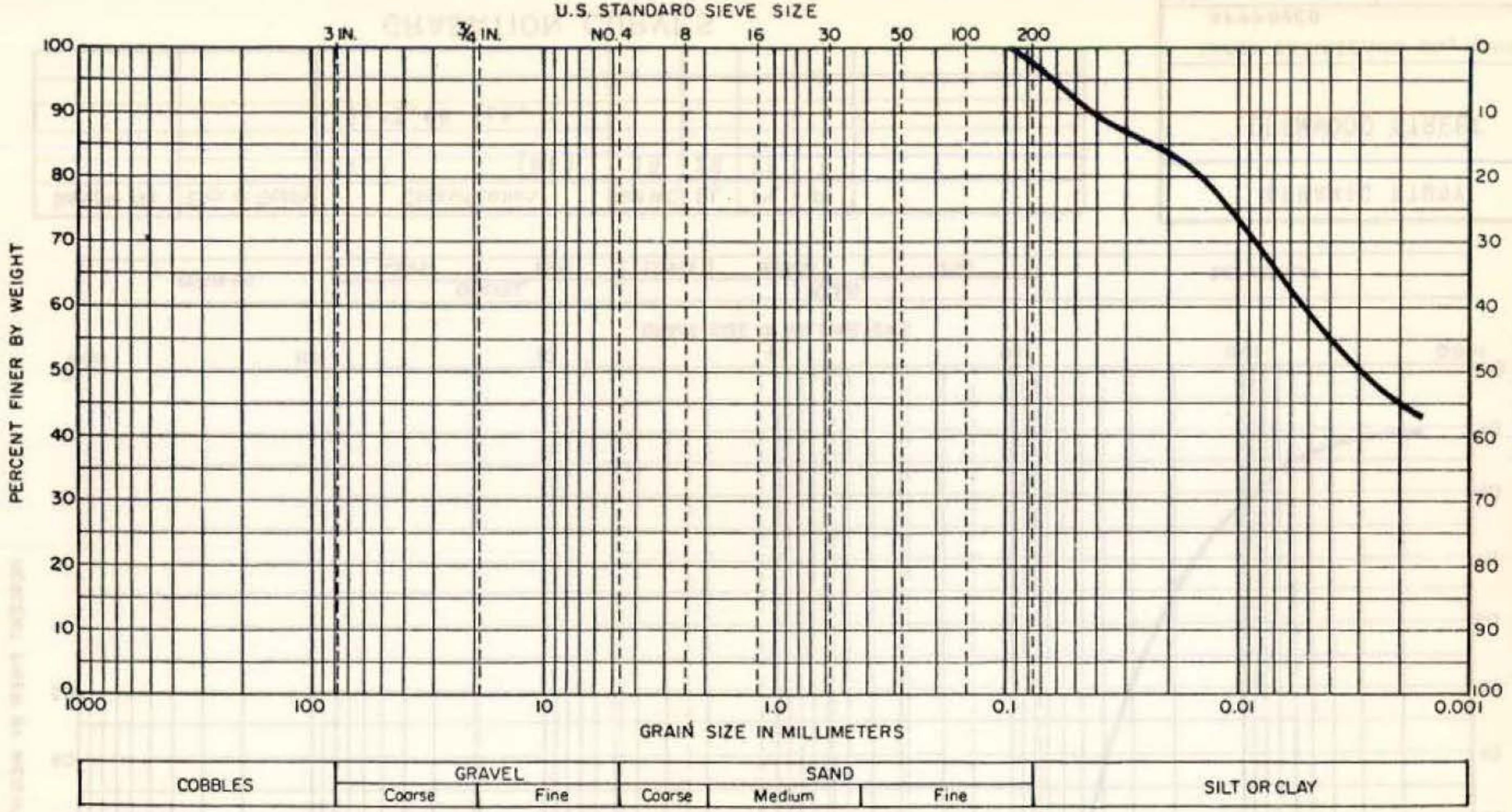
OPENWOOD STREET

HARZA ENGINEERING CO., CHICAGO

APPROVED.........................

DATE

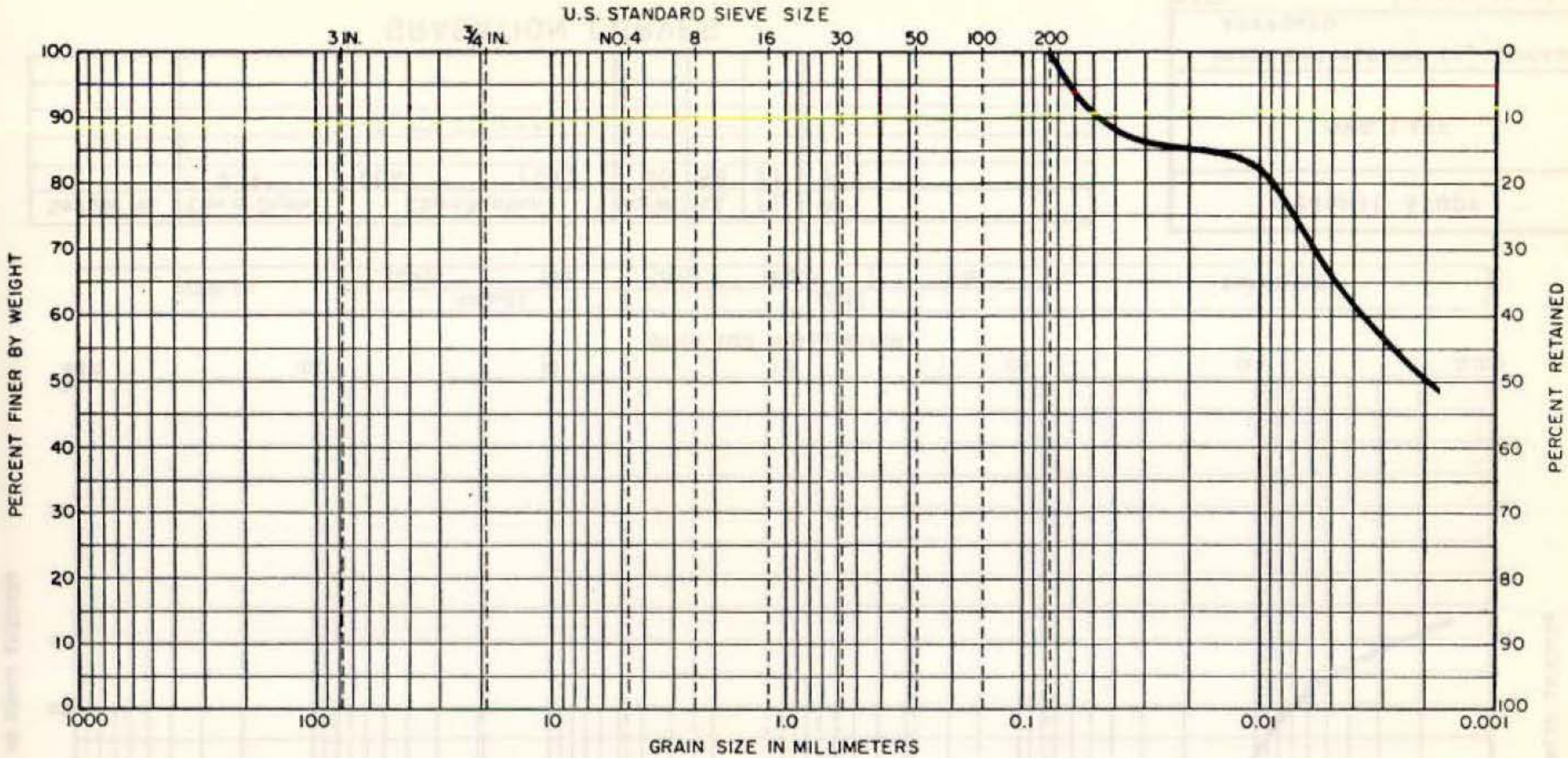
FIGURE IV-A-1



Sample No.	Elev. or Depth	Classification	Nat WC	LL	PL	PI	
	18.5-49.5'	CLAY (CH)	52	70	26	44	
		G_s=2.70 est.					

GRADATION CURVES

CERAMIC STUDY
OZARK REVETMENT
HARZA ENGINEERING CO., CHICAGO APPROVED..........................
DATE / FIGURE IV-A-2

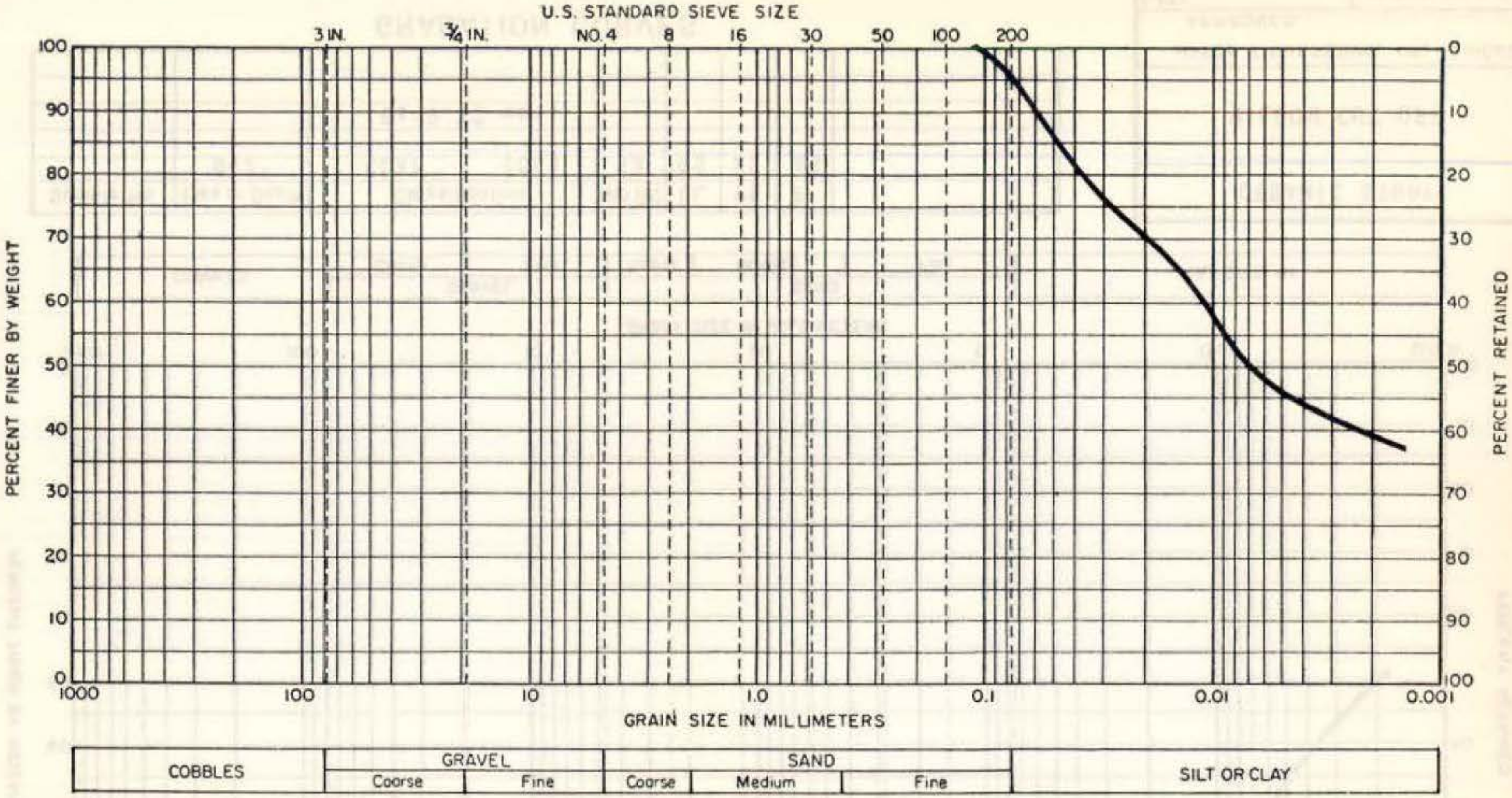


COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Elev. or Depth	Classification	Nat WC	LL	PL	PI	
	0-2'	CLAY (CH)	43	95	32	63	
		Gs=2.72 est.					

GRADATION CURVES

CERAMIC STUDY	
WILLOW CUT-OFF	
HARZA ENGINEERING CO., CHICAGO APPROVED........................	
DATE	FIGURE IV-A-3



Sample No.	Elev. or Depth	Classification	Nat WC	LL	PL	PI	
	0.4'	CLAY (CH)	30	58	21	37	
		G_s=2.70 est.					

GRADATION CURVES

CERAMIC STUDY

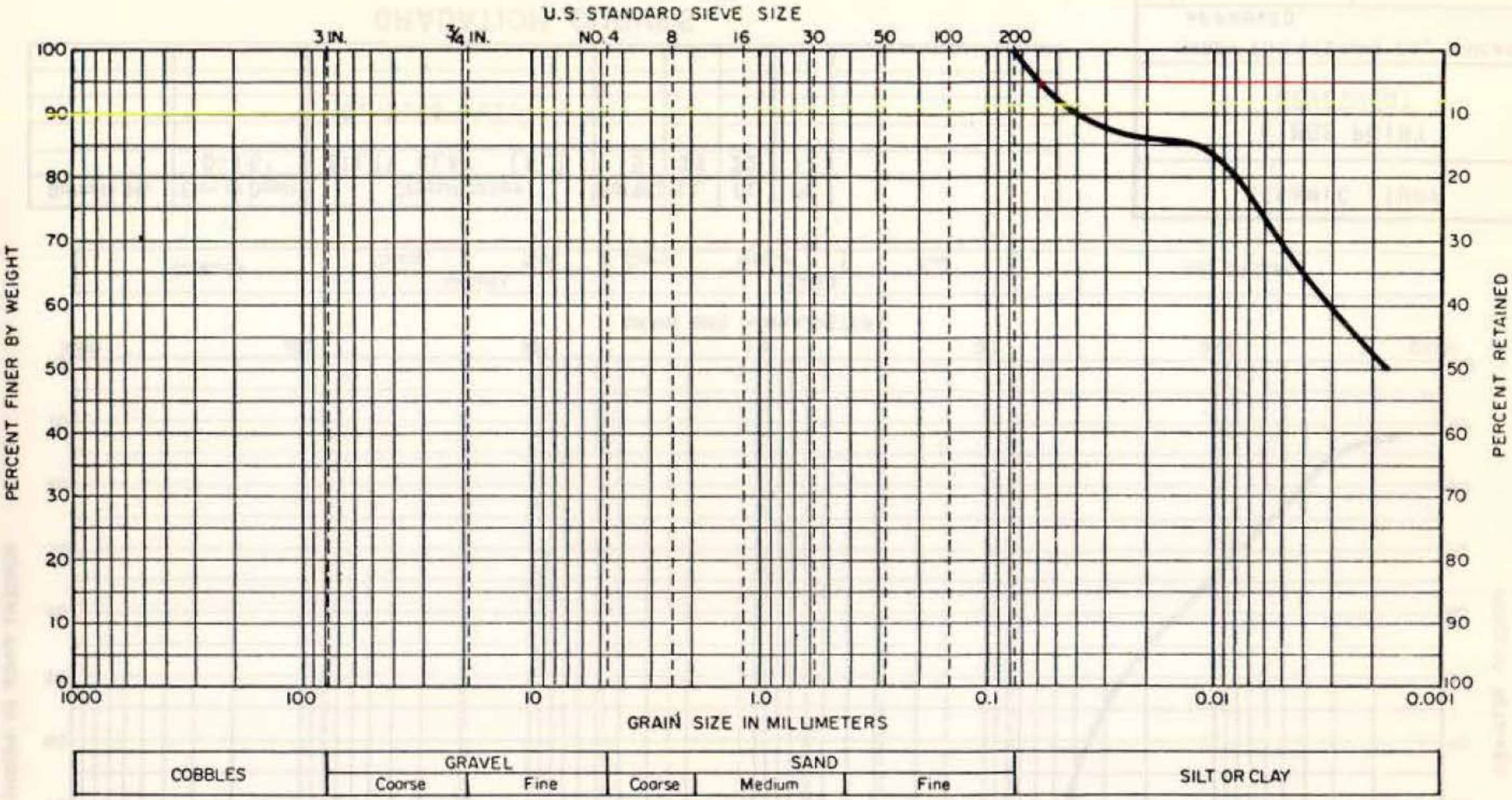
LONG LAKE

HARZA ENGINEERING CO., CHICAGO

APPROVED.........................

DATE

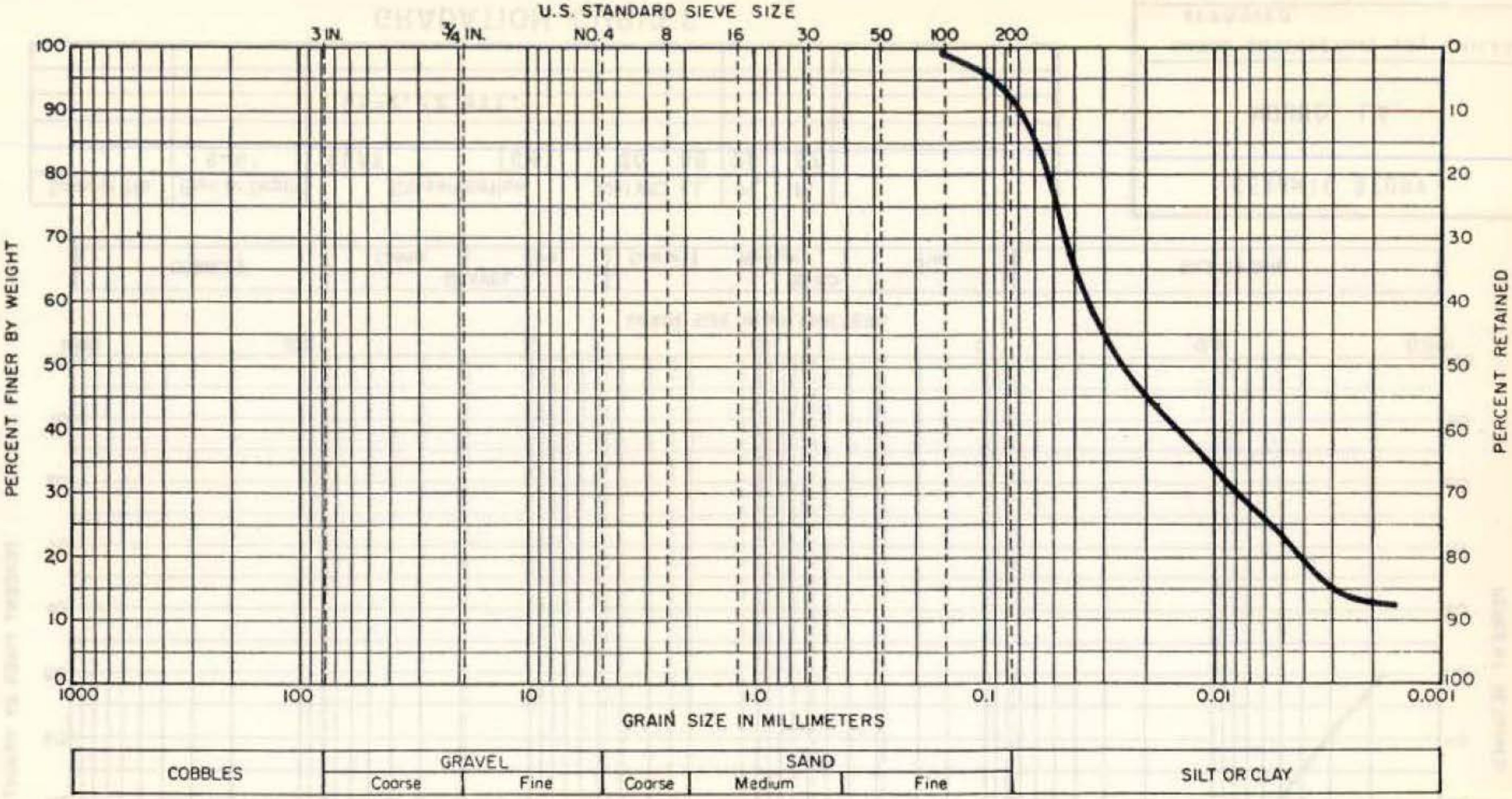
FIGURE IV-A-4



Sample No.	Elev. or Depth	Classification	Nat WC	LL	PL	PI
	3-4'	CLAY (CH)	30	86	29	57
		Gs=2.72 est.				

GRADATION CURVES

CERAMIC STUDY
MOUND, LA.
HARZA ENGINEERING CO., CHICAGO APPROVED..........................
DATE / FIGURE IV-A-5



Sample No.	Elev. or Depth	Classification	Nat WC	LL	PL	PI	
	0-15'	SILTY CLAY (CL)	15	33	22	11	
		G_S=2.68 est.					

GRADATION CURVES

CERAMIC STUDY

KINGS POINT
REVETMENT

HARZA ENGINEERING CO., CHICAGO

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DATE

FIGURE IV-A-6

Table IV-A-1

INFORMATION ON SOIL SAMPLES FOR RIVER BANK STABILIZATION CERAMICS STUDY

Type of Deposit	Location of Deposit	Depth Sampled ft.	LL** %	PL** %	PI**	Approx. Natural Water Content %	Processed Water Content %	Classification*
Loessial	Vicksburg, Mississippi; vicinity of Openwood Street	-	28	23	5	14	13.8	Silt ML
Clay Plug	Ozark Revetment; west bank of Mississippi River; mile 569-573	18.5-49.5	70	26	44	52	47.4	Clay CH
Clay Plug	Willow Cutoff; west bank of Mississippi River; mile 457	0 - 2***	95	32	63	43	29.2	Clay CH
Backswamp	Long Lake Area; approx. latitude 90°53.5', approx. longitude 32°24'; 3 1/2 miles NW Vicksburg	0 - 4	58	21	37	30	16.2	Clay CH
Backswamp	Mound, La.; approx. longitude 91°00', approx. latitude 32°19.4'; 8 miles SW Vicksburg	3 - 4	86	29	57	30	27.0	Clay CH
Overburden	Kings Point Revetment; east bank of Mississippi River; mile 434-435	0 - 15	33	22	11	15	13.4	Silty Clay CL

* Corps of Engineers' Unified Soil Classification System

** LL = liquid limit; PL = plastic limit; and PI = plasticity index

*** Samples taken on sloping river bank

Table IV-A-2

PHYSICAL PROPERTIES OF CLAYS

(Conventional Processing)

Sample	Bag Moisture (%)	Extruded Moisture (%)	Total Shrinkage (%) 05	01	5	Bulk Density (psi) 05	01	5	Percent Absorption 05	01	5
Loessial (Openwood Street)	1.9	20.5	0.0	2.5	8.9	1.39	1.46	2.12	34.9	32.1	4.5
Clay Plug (Ozark Revetment)	47.5	47.5	11.2	bloated		1.60	1.13	-	15.7	41.0	-
Clay Plug (Willows Cut-off)	21.0	25.8	9.0	bloated		1.48	1.29	-	20.2	28.3	-
Backswamp (Long Lake)	10.3	20.2	8.7	bloated		1.68	1.01	-	17.8	53.6	-
Backswamp (Mound, La.)	22.9	24.8	broken & bloated			-	-	-	-	-	-
Overburden (King's Point)	7.2	19.5	6.1	11.3	8.1	1.78	2.23	1.65	16.6	1.8	7.7

Table IV-A-3

PHYSICAL PROPERTIES OF CLAYS

(Hot Pressed)

Sample	Forming Pressure (psi)	Max. Temp. (°F)	High Temp. Range (°F)	Time at High Temp. Range (min.)	Crushing Strength (psi) Low	Max.	Ave.	Density (g/cc) (lbs/ft^3)	Absorption (%)
Loessial (Openwood Street)	3,800	2,190	2,120-2,190	3.50	6,666	15,459	13,211	2.30 (144)	5.23
Clay Plug (Ozark Revetment)									
Low Glass	3,800	1,985	1,830-1,985	1.40	2,334	4,765	3,171	2.00 (125)	11.90
High Glass	3,800	1,970	1,830-1,975	1.70	10,297	28,881	16,540	2.62 (164)	3.26
Clay Plug (Willow Cutoff)									
	3,800	1,885	1,780-1,885	1.45	5,774	9,008	7,080	1.74 (109)	19.60
	2,550	1,960	1,830-1,960	1.85	5,160	12,653	7,367	1.97 (123)	11.91
Backswamp (Long Lake)	3,800	1,960	1,830-1,960	1.40	9,749	27,114	16,892	2.35 (148)	3.82
Backswamp (Mound, La.)	3,800	1,960	1,830-1,960	1.55	4,484	11,200	6,582	2.20 (138)	3.84
Overburden (King's Point)	3,800	2,155	1,920-2,155	2.75	8,373	17,609	11,982	2.32 (145)	3.46

Table IV-A-4

EXPERIMENTAL DATA ON HOT PRESSED SAMPLES

Sample No.		Forming Pressure (psi)	Maximum Temperature (^oF)	Crushing Strength (psi)	Density (g/cm^3)	Absorption (%)
		LOESSIAL (OPENWOOD STREET)				
1		3800	2185	13,944	2.38	3.83
2			2230	6,666	2.23	5.93
3			mold broke	-	-	-
4			2165	15,127	2.25	6.94
5			2185	15,459	2.30	6.07
6			mold broke	-	-	-
7			2190	14,861	2.36	3.39
8			mold broke	-	-	-
9			mold broke	-	-	-
	Average:			13,211	2.30	5.23
1a		3800	2105	no tests	1.72	20.8
2a			2105	no tests	1.82	19.0
3a			2185	no tests	1.88	16.5
		CLAY PLUG (OZARK REVETMENT)				
Low Glass						
1		3800	1950	2,334	1.85	13.2
2			1990	cracked	-	-
3			1955	2,970	1.87	12.9
4			1955	2,825	1.90	11.2
5			1920	2,620	1.92	12.5
7			1990	4,765	2.44	9.5
	Average:			3,171	2.00	11.9
High Glass						
6		3800	1080	10,297	2.93	3.26
8			1950	10,442	2.31	3.26
9			1990	28,881	-	-
	Average:			16,540	2.62	3.26

Table IV-A-4

EXPERIMENTAL DATA ON HOT PRESSED SAMPLES (cont'd.)

Sample No.	Forming Pressure (psi)	Maximum Temperature (^oF)	Crushing Strength (psi)	Density (g/cm^3)	Absorption (%)
		CLAY PLUG (WILLOW CUT-OFF)			
High Pressure					
1	3800	specimen broke	-	-	-
2		1845	6,142	1.81	9.20
3		specimen broke	-	-	-
4		1840	5,952	1.76	9.43
5		1830	8,535	1.92	5.67
6		specimen broke	-	-	-
7		1830	5,774	1.91	5.63
8		1860	-	1.75	9.63
9		1860	9,008	1.41	6.33
Average:			7,080	1.76	7.65
Low Pressure					
1	2550	2100	7,376	2.01	4.37
2		2110	5,160	2.01	4.16
3		2125	12,653	2.02	4.04
4		2120	7,064	1.94	4.97
5		2120	2,580*	1.97	3.28
6		2125	6,302	2.07	1.72
7		2125	-	1.64	17.56*
8		2125	6,695	1.94	4.72
9		2135	8,231	1.86	8.75
10		2125	5,879	1.75	10.86
11		2120	6,946	1.88	8.03
Average:			7,367	1.97	5.49
		BACKSWAMP (LONG LAKE)			
1	3800	1965	9,749	2.12	8.16
2		1950	10,458	2.35	3.33
3		1960	15,479	2.38	3.73
4		1930	13,663	2.40	2.94

Table IV-A-4

EXPERIMENTAL DATA ON HOT PRESSED SAMPLES (cont'd.)

Sample No.		Forming Pressure (psi)	Maximum Temperature (^oF)	Crushing Strength (psi)	Density (g/cm^3)	Absorption (%)
			BACKSWAMP (LONG LAKE) cont'd.			
5*			1950	3,194	2.08	8.71
6			1930	22,542	2.46	2.17
7			1945	27,114	2.45	2.22
8			1960	16,329	2.30	5.07
9			1930	19,802	2.38	3.0
10*			1945	1,658	2.08	8.78
	Average:			16,892	2.35	3.82
			BACKSWAMP (MOUND)			
1		3800)				
2		)		First four samples were not satisfactory for testing		
3		)				
4		)				
5			1900	5,483	2.49	1.25
6			1920	11,200	2.20	1.73
7			1940	2.088	2.06	6.88
8*			1940	609	1.94	11.6
9*			1960	5,160	2.07	6.51
10			1940	4,484	2.04	5.88
	Average:			6,582	2.20	3.84
			OVERBURDEN (KING'S POINT)			
1		3800	2005	-	2.10	8.51
2			2075	10,358	2.33	2.59
3			2125	12,899	2.41	2.88
4			2120	10,114	2.40	3.28
5			2145	11,516	2.24	4.92
6			2125	8,373	2.42	3.09
7			2135	10,114	2.56	1.11
8			2125	17,609	2.44	2.82
9			2165	11,576	2.16	3.58
10			2135	15,110	2.45	2.57
11			2150	12,247	2.29	2.67
	Average:			10,901	2.32	3.46

* Data not included in averages

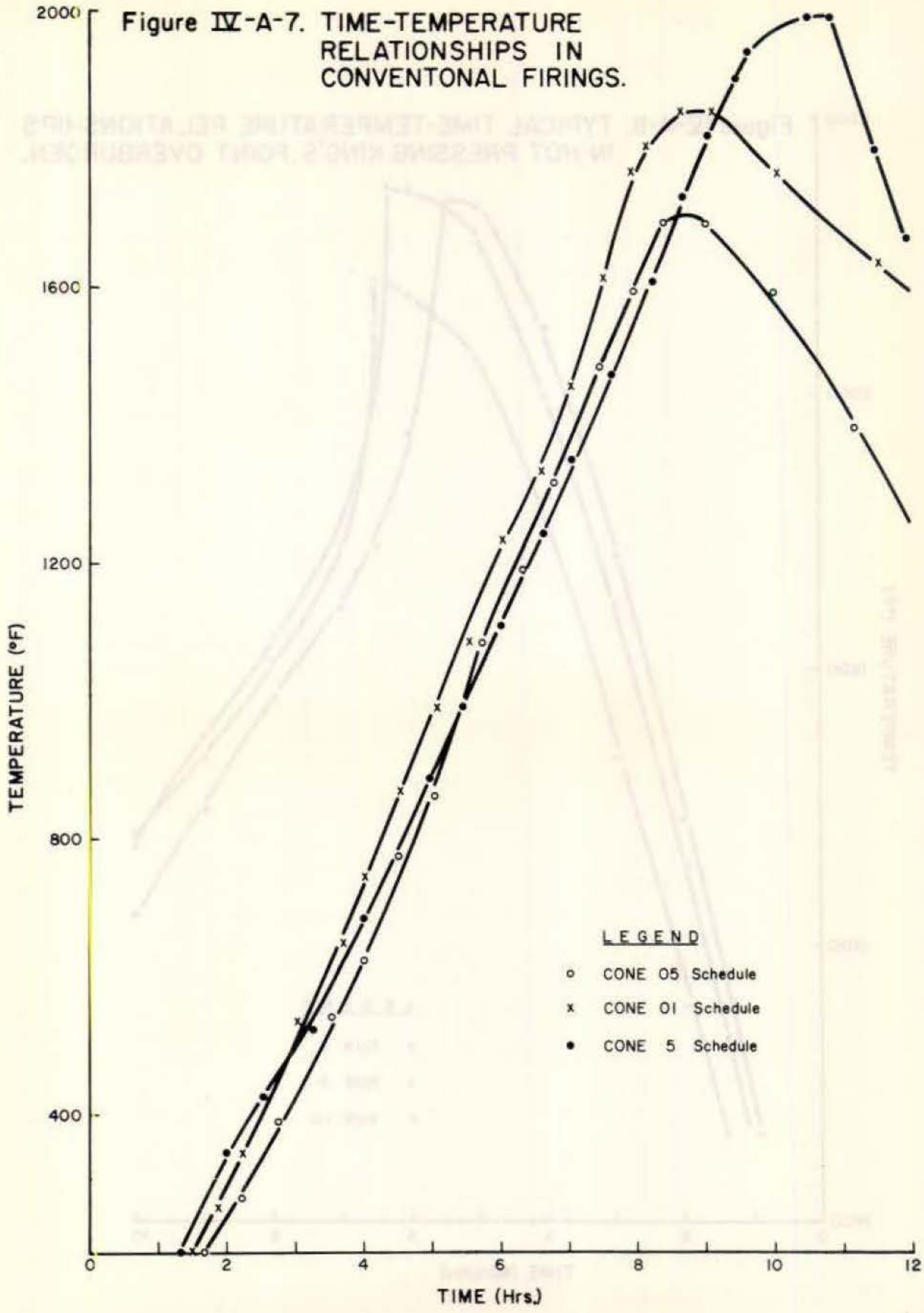
Figure IV-A-7. TIME-TEMPERATURE RELATIONSHIPS IN CONVENTONAL FIRINGS.
2000
1600
1200
800
400
TEMPERATURE (°F)
0
2
4
6
8
10
12
TIME (Hrs.)
LEGEND
o CONE 05 Schedule
x CONE 01 Schedule
• CONE 5 Schedule

Figure IV-A-8. TYPICAL TIME-TEMPERATURE RELATIONSHIPS IN HOT PRESSING KING'S POINT OVERBURDEN.

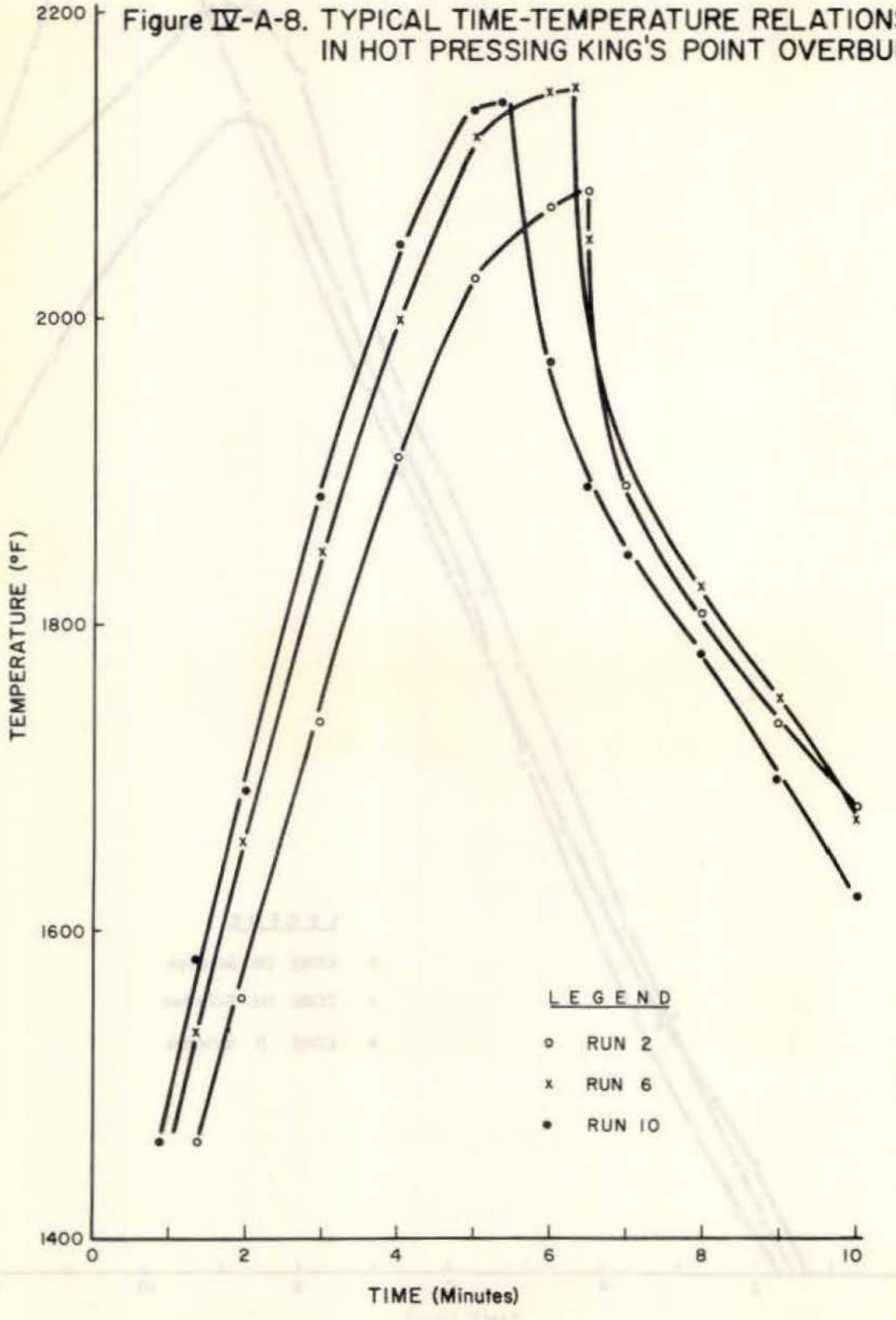

Fig. IV-A-9. Graphite die, rammer (being held), and typical sample after extrusion (sitting on die).

Fig. IV-A-10. Taking temperatures with an optical pyrometer during a hot pressing.

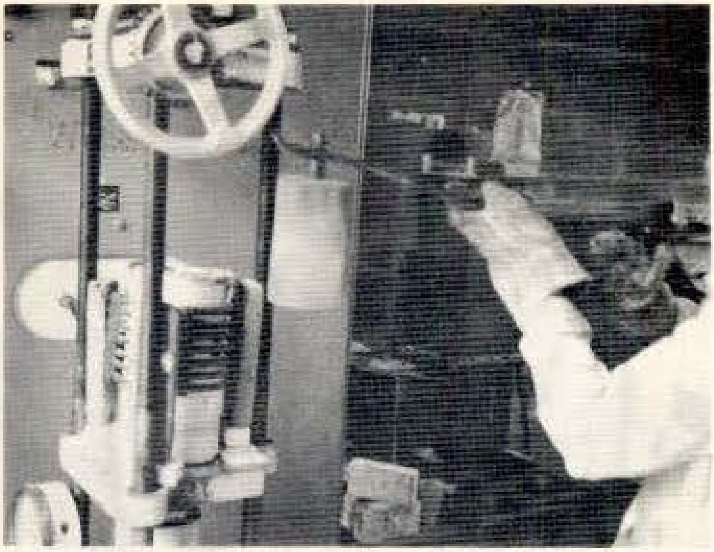

Fig. IV-A-11. Removing the hot die immediately after pressing.

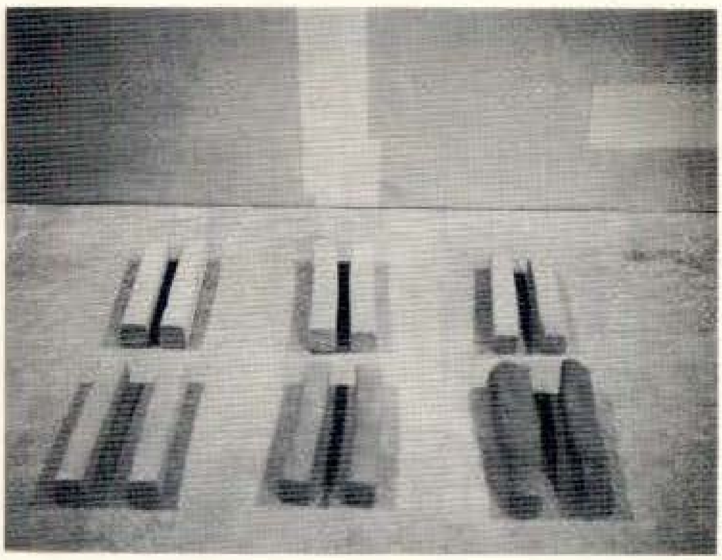

Fig. IV-A-12. Test samples fired by conventional methods. Front row: silt overburden from King's Point Revetment; Back row: loess from Vicksburg bluffs. Left to right: samples fired at cone 05, 01, and 5.

Fig. IV-A-15. Conventionally formed samples.

Fig. IV-A-16. Hot-pressed formed samples after compressive strength test.

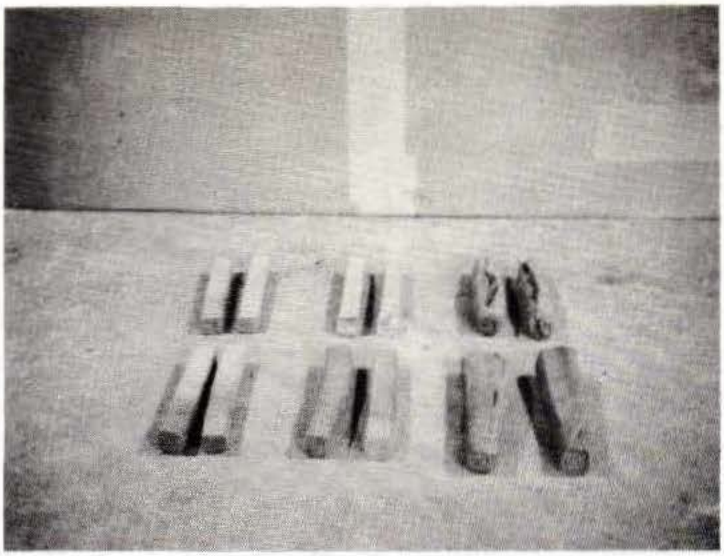

Fig. IV-A-13. Test samples of clay plug deposits fired by conventional methods. Front row: Willow Cutoff samples. Back row: Ozark Revetment samples. Left to right: samples fired at cone 05, 01, and 5.

Fig. IV-A-14. Test samples of backswamp deposits fired by conventional methods. Front row: Mound, Louisiana samples. Back row: Long Lake area samples. Left to right: samples fired at cone 05, 01, and 5.

PART V

POINT BAR STABILIZATION

Table of Contents

PART V

POINT BAR STABILIZATION

Introduction

Point bars are the sand deposits which build up on the convex bends of river meanders as the bend migrates outward.

Only minor efforts have been made on the Mississippi River to stabilize these deposits, however, greater control could be desirable. Flood flows tend to cut across the point bars and scour off the upstream surface. The sand is then deposited in the downstream crossing, resulting in a reduction in the channel depth. Thus, if the point bars were stabilized, less material would be deposited in the crossings and the problem of maintaining a navigable channel through the crossings would be reduced. Brief consideration has been given to the possibility of using some of the cheaper methods investigated for the concave banks to stabilize the point bars.

The design criteria for point bar stabilization differs somewhat from the design criteria for concave bank stabilization. The point bar protection would have to be durable, erosion resistant, and capable of adjusting to minor foundation settlement, but probably would not have to be as permeable (though it must retain the foundation material). The area to receive protection will be composed of fine to medium sand (no clay), and will have flat, uniform natural slopes. The area to be protected will be partially submerged during low flow; however, the major portion will be exposed. Experimental installations are needed to determine how much of the point bar area must be stabilized to achieve the desired results. In previous installations only the upstream portion of a point bar was protected. Stabilization methods, to be economically justifiable under present conditions, would have to be much less expensive than those used on the concave banks. For the purpose of this study an arbitrary maximum cost of \$0.10 per square foot (in place) was chosen as the controlling cost.

Many of the methods and materials discussed in previous parts of this report might be technically suitable, but too expensive, for point bar stabilization. Other methods and materials which are technically suitable and also meet the economic criteria will require some modification for application on point bars. Other materials and methods not previously discussed will be covered in more detail.

Riprap

Riprap has been discussed previous (see Chapter 1, Part III). The flatter slopes of the point bars will allow the use of a smaller riprap than required on the upper bank. Bertram reports (Ref. V-1) that in surveying the effectiveness of slope protection on large dams, it was found that a 12-inch layer of well graded gravel with a maximum size of three inches provided effective protection against waves up to eight feet in height for slopes of 8h:1v to 15h:1v on earth embankments. It is believed that this criterion or a modification thereof would also apply to point bar stabilization.

Thus, it might be possible to provide sufficient protection by placing a layer of natural stone "quarry run" spalls of sufficient gradation to eliminate the need for separate filter and riprap layers.

Monolithic Protection

Most of the methods of monolithic protection discussed in Chapter 2, Part III, and in Chapters 2 and 3, Part IV, could also be used on the point bars. Because of economic requirements, it would probably not be feasible to use such materials in any of the more complicated forms such as reinforced or sand filled mattresses.

Synthetic Sheeting and Fabrics: A single layer of synthetic sheeting or fabric might provide adequate protection if secured with anchors or randomly placed riprap. The elastomers would probably be more suitable for such use than the plastics because of their greater resistance to actinic deterioration (see Chapter 2, Part IV). If an asphalt or PRR compound sheeting were used, the cost of the materials would be about $0.05 per square foot (excluding ballast or anchors).

Asphalt: The general low cost of asphaltic materials make them especially attractive for use in stabilizing the point bars. Some of these materials already described under upper bank monolithic protection (pages III-23 to III-29) such as cationic emulsion spray and cationic emulsion "solid slurry" might be used for stabilizing the portions which are not submerged. For those portions which are submerged it might be possible to use one of several methods developed for sealing canals and reservoirs without dewatering.

The first of three methods involves the use of one of several sealing agents which are mixed with the water and allowed to seep into the foundation. They can be formulated so that they either deposit in one layer at some distance below the surface, or uniformly through the soil. In order to prevent such agents from being carried away by the current if used on the Mississippi it would be necessary to confine them in a plastic or metal pocket against the bank. The depth to which these agents permeate is mainly dependent upon three factors, the particular formulation, the grain size of the soil, and the direction of the rate for seepage.

Under favorable conditions, penetration can be expected to be as great as 30 inches. It is suspected, however, that there is insufficient seepage into the point bars to achieve any more than superficial stabilization.

The materials used in this method of seepage control cost from \$0.15 to \$0.20 per gallon and the normal application rate is about one-half gallon per square yard. Experimental installations are reported to have cost about \$0.03 per square foot in place (Ref. III-2).

A method similar to the above but probably requiring less time, is one that the Armour Industrial Chemical Company has experimented with for controlling canal seepage. In their experiments a section of canal was closed off and filled with a diluted cationic asphalt emulsion. The sand on the bottom was then stirred up mechanically, and while it was in suspension, bits of asphalt adhered to the individual grains. Upon settling, the asphalt tended to bind the grains together and reduce the permeability. An apparatus which might be used in an adaptation of this method is illustrated in Figure V-1. An inverted pan is moved across the area to be protected. As it moves a diluted cationic emulsion is injected into it under pressure in such a way that the jets stir up the soil. The asphalt globules attach themselves to the suspended sand particles and bind them together as they settle. As the stern of the apparatus passes over the settled mixture, it levels and slightly compacts it. The cost of such an apparatus capable of producing the required depth of stabilization is unknown. The cationic asphalt emulsion costs approximately \$0.16 per gallon.

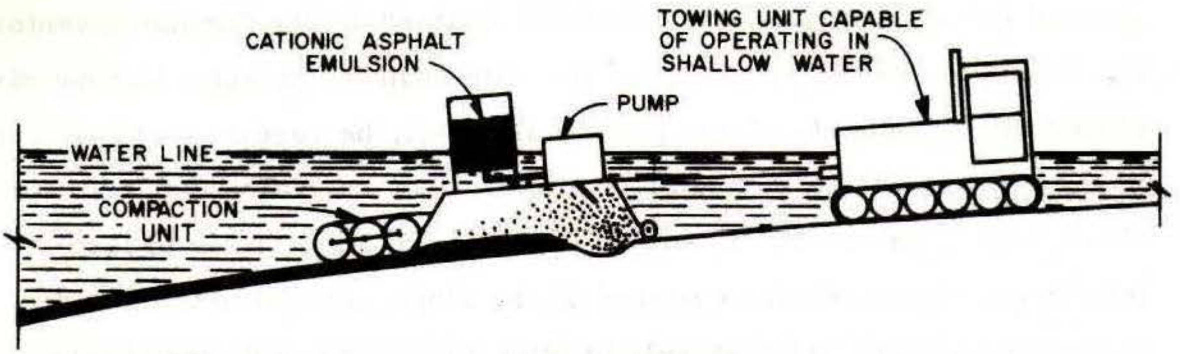

Figure V-1. CATIONIC ASPHALT EMULSION DISTRIBUTOR.

A third method of low cost canal sealing with which the Bureau of Reclamation is experimenting (Ref. III-2) injects cationic emulsions horizontally into the subgrade at a small distance below the surface through an apparatus similar to an agricultural harrow. Laboratory experiments have demonstrated that a stabilized (and impermeable) layer of soil can be produced below the surface by this method. Although field applications have so far been unsuccessful, they are hoping for better results in another experimental application to be conducted this year. The cost of materials is approximately $0.16 per gallon.

A fourth method of using asphalt to stabilize the point bar subaqueous slopes would be to distribute a layer of asphalt mastic on the surface. An apparatus similar to that described on pages IV-68 to IV-70 and illustrated in Figure IV-8 could be used in such an application. Such an application developed for use on point bars probably would not have to be as complicated as one developed for use in protecting the lower banks because of the smaller depths, lower velocities, and flatter slopes. The deposited layer would have to be fairly thick, however, to resist being carried away by the current.

Chemicals: A relatively new type of material which has been developed for sealing canals is called "Hydraton" by its German inventor, Dr. K. F. G. Keil (Ref. V-2, 3, 4). Although the inventor has not disclosed the ingredients of this product in detail, he reports that it is composed of a special sodium silicate preparation, special clays, and other chemicals. These are mixed together in a concrete mixer to form a thixotropic slurry which is poured on the slope, spread to the desired thickness by hand, and then gels to form an impermeable membrane. Contact was made with the firm in this country which has the rights to this process, the Chemical Soil Solidification Company. Through them, with Dr. Keil and the construction firm of Gebhardt & Koenig in Germany (who have installed this protection on a shipping canal, the Rhein Main Danube Canal), the suitability of this process for riverbank protection was evaluated.

Because the mixture is inherently impermeable, it would not be suitable for protection of the concave banks. The material does not appear to be particularly erosion resistant. It has apparently required some protective cover in former installations and would certainly require some on the Mississippi. The material is not suitable for use on exposed surfaces because it undergoes great volume changes upon drying. The minimum cost of the preparation in place is estimated to be $15.50 per cubic yard, which, combined with the above physical factors, eliminates it from being considered for either point bar or concave bank protection on the lower Mississippi River.

PART V

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GLOSSARY

AND

GENERAL BIBLIOGRAPHY

GLOSSARY

actinic deterioration - Deterioration of a material through chemical change caused by ultraviolet light.

asphalt mastic - A mix of fine mineral aggregate, fillter and bitumen, with or without chippings, all in such proportions that the mix can be applied hot by pouring into place.

breaking - The process of asphalt globules going from an emulsion and becoming attached to aggregate.

ceramic - A material composed mainly of soil that has been formed into a shape by bonding or fusing soil grains together with the application of heat and/or pressure.

cones - Slender, trigonal pyramids composed of ceramic materials that are used in the firing of ceramics to determine the combined effect of temperature and time.

elastomer - A material, usually synthetic, having elastic properties akin to those of rubber.

fabric - A porous material woven from strands of material.

filling - The threads across the width of a fabric as it is weaved.

incipient fusion - In ceramics, the state of the soil when it is just beginning to change from a solid to a liquid state.

laser rays - A highly concentrated beam of light formed by radiation emitted from atoms as electrons drop back to lower energy states from the higher energy states that they have been stimulated to by various techniques.

mono-filament yarn - Strands of fibrous material in which each filament is the strand used to weave a fabric.

multi-filament yarn - Strands of fibrous material in which filaments are spun into strands that are woven into fabrics.

morphology - The study of the shape and contours of the surface of the earth.

pyro-plastic - Method of forming ceramic materials by a combination of heat and pressure.

resins - Organic substances exuded from various plants or prepared synthetically.

sheeting - Sheets of any thickness of synthetic materials, formed as a non-porous, continuous material mass.

stream power - Product of the mean stream velocity and shear stress on a section.

stripping - Process by which asphalt is detached from the aggregate.

synthetic - Artificial, not derived immediately from a natural product.

synthetic fiber - Any substance that can be separated into threads or threadlike structures for weaving fabrics.

synthetic films - A non-porous continuous sheet of synthetic material up to one-half inch in thickness.

thermal diffusivity - A measure of the rate at which matter adjusts to a temperature change.

thermite process - Intense heat is liberated by the exothermic reaction of an ignited aluminum powder and metal oxide mixture. The aluminum is oxidized and the metallic oxides are reduced.

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